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	Engineering and Design TUNNELS AND SHAFTS IN ROCK	
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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Manual
No. 1110-2-2901

30 May 1997

Engineering and Design
TUNNELS AND SHAFTS IN ROCK

1. Purpose. This manual was prepared by CECW-ED and CECW-EG and provides technical criteria and guidance for the planning, design, and construction of tunnels and shafts in rock for civil works projects. Specific areas covered include geological and geotechnical explorations required, construction of tunnels and shafts, design considerations, geomechanical analysis, design of linings, and instrumentation and monitoring.

2. Applicability. This manual applies to all Headquarters, U.S. Army Corps of Engineers (HQUSACE) elements, major subordinate commands, districts, laboratories, and field-operating activities having responsibilities for the design of civil works projects.

FOR THE COMMANDER:

OTIS WILLIAMS
Colonel, Corps of Engineers
Chief of Staff

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Chapter 1
Introduction

1-1. Purpose

The purpose of this manual is to provide technical criteria and guidance for the planning, design, and construction of tunnels and shafts in rock for civil works projects. Specific areas covered include geological and geotechnical explorations required, construction of tunnels and shafts, design considerations, geomechanical analysis, design of linings, and instrumentation and monitoring.

1-2. Scope

a. This manual presents analysis, design, and construction guidance for tunnels and shafts in rock. A team comprised of highly skilled engineers from many disciplines is required to achieve an economical tunnel or shaft design that can be safely constructed while meeting environmental requirements. The manual emphasizes design, construction and an understanding of the methods, and conditions of construction essential to the preparation of good designs.

b. Since construction contracting is a major consideration in underground construction, the manual discusses some of the basic issues relating to contract document preparation; however, contract preparation is not covered.

c. The procedures in this manual cover only tunnels and shafts in rock. The general design philosophy and construction methods for rock tunnels and shafts is vastly different than for tunnels or shafts in soft ground. Therefore, tunnels and shafts in soft ground is not covered by this manual.

d. There are many important nontechnical issues relating to underground construction such as economics, as well as issues of operation, maintenance, and repair associated with the conception and planning of underground projects. These issues are not covered by this manual.

1-3. Applicability

This manual applies to all Headquarters, U.S. Army Corps of Engineers (HQUSACE) elements, major subordinate commands, districts, laboratories, and field-operating activities having responsibilities for the design of civil works projects.

1-4. References

Required and related publications are listed in Appendix A.

1-5. Distribution Statement

Approved for public release, distribution is unlimited.

1-6. Terminology

Appendix B contains definitions of terms that relate to the design and construction of tunnels and shafts in rock.

Chapter 2
General Considerations

2-1. Approach to Tunnel and Shaft Design and Construction

Design and construction of tunnels and shafts in rock require thought processes and procedures that are in many ways different from other design and construction projects, because the principal construction material is the rock mass itself rather than an engineered material. Uncertainties persist in the properties of the rock materials and in the way the rock mass and the groundwater will behave. These uncertainties must be overcome by sound, flexible design and redundancies and safeguards during construction. More than for any other type of structure, the design of tunnels must involve selection or anticipation of methods of construction.

2-2. Rock as a Construction Material

a. When a tunnel or shaft is excavated, the rock stresses are perturbed around the opening and displacements will occur. The rock mass is often able to accommodate these stresses with acceptable displacements. The stable rock mass around the opening in the ground, often reinforced with dowels, shotcrete, or other components, is an underground structure, but a definition of the degree of stability or safety factor of the structure is elusive.

b. If the rock is unstable, rock falls, raveling, slabbing, or excessive short- or long-term displacements may occur and it must be reinforced. This can be accomplished either by preventing failure initiators such as rock falls or by improving the ground's inherent rock mass strength (modulus). Either way, the rock mass, with or without reinforcement, is still the main building material of the tunnel or shaft structure.

c. Unfortunately, geologic materials are inherently variable, and it is difficult to define their properties with any certainty along a length of tunnel or shaft. In fact, most tunnels must traverse a variety of geologic materials, the character of which may be disclosed only upon exposure during construction. Thus, ground reinforcement and lining must be selected with adaptability and redundant characteristics, and details of construction must remain adaptable or insensitive to variations in the ground.

d. Geologic anomalies and unexpected geologic features abound and often result in construction difficulties or risks to personnel. For example, inrush of water or occurrence of gases can cause great distress, unless the contractor is prepared for them. Thus, an essential part of explorations and design revolves around defining possible and probable occurrences ahead of time, in effect, turning the unexpected into the expected. This will permit the contractor to be prepared, thus improving safety, economy, and the duration of construction. In addition, differing site condition claims will be minimized.

2-3. Methods and Standards of Design

a. Considering the variability and complexity of geologic materials and the variety of demands posed on finished underground structures, it is not surprising that standards or codes of design for tunnels are hard to find. Adding to the complexity is the fact that many aspects of rock mass behavior are not well understood and that the design of man-made components to stabilize the rock requires consideration of strain compatibility with the rock mass.

b. This manual emphasizes methods to anticipate ground behavior based on geologic knowledge, the definition of modes of failure that can, in many cases, be analyzed, and principles of tunnel design that will lead to safe and economical structures, in spite of the variability of geologic materials.

2-4. Teamwork in Design

a. Because of the risks and uncertainties in tunnel and shaft construction, design of underground structures cannot be carried out by one or a few engineers. Design must be a careful and deliberate process that incorporates knowledge from many disciplines. Very few engineers know enough about design, construction, operations, environmental concerns, and commercial contracting practices to make all important decisions alone.

b. Engineering geologists plan and carry out geologic explorations, interpret all available data to ascertain tunneling conditions, and define geologic features and anomalies that may affect tunnel construction. Engineering geologists also participate in the design and assessment of ground support requirements, initial ground support, the selection of remedial measures dealing with anomalous conditions, selection of lining type, and the selection of basic tunnel alignment. The engineering geologist may require the help of geohydrologists or other specialists. Note: details of initial ground support design are usually left to the contractor to complete.

c. Hydraulics engineers must set the criteria for alignment and profile, pressures in the tunnel, and tunnel finish (roughness) requirements and must be consulted for analysis and opinion when criteria may become compromised or when alternative solutions are proposed.

d. Structural engineers analyze steel-lined pressure tunnels and penstocks and help analyze reinforced concrete linings. Structural engineers also assist in the basic choices of tunnel lining type and participate in the selection and design of initial ground support components such as steel sets.

e. Geotechnical engineers participate in the design and assessment of ground support requirements, initial ground support, the selection of remedial measures dealing with anomalous conditions, selection of lining type, and basic tunnel alignment.

f. Civil engineers deal with issues such as construction site location and layout, drainage and muck disposal, site access, road detours, and relocation of utilities and other facilities.

g. Civil engineers or surveyors prepare base maps for planning, select the appropriate coordinate system, and establish the geometric framework on which all design is based as well as benchmarks, criteria, and controls for construction.

h. Environmental staff provides necessary research and documentation to deal with environmental issues and permit requirements. They may also lead or participate in public involvement efforts.

i. Construction engineers experienced in underground works must be retained for consultation and review of required or anticipated methods of construction and the design of remedial measures. They also participate in the formulation of the contract documents and required safety and quality control plans.

j. Other professionals involved include at least the specification specialist, the cost estimator (often a construction engineer), the drafters/designers/computer-aided drafting and design (CADD) operators, and the staff preparing the commercial part of the contract documents.

2-5. The Process of Design and Implementation

Aspects of tunnel engineering and design, geology, and geotechnical engineering must be considered in all stages of design. The following is an overview of the design and implementation; details are discussed in later sections of this manual.

a. Reconnaissance and conception. Project conception in the reconnaissance stage involves the identification and definition of a need or an opportunity and formulation of a concept for a facility to meet this need or take advantage of the opportunity. For most USACE projects with underground components, the type of project will involve conveyance of water for one purpose or another—hydropower, flood control, diversion, water supply for irrigation or other purposes.

b. Feasibility studies and concept development.

(1) Activities during this phase concentrate mostly on issues of economy. Economic feasibility requires that the benefits derived from the project exceed the cost and environmental impact of the project. Design concepts must be developed to a degree sufficient to assess the cost and impact of the facility, and "show-stoppers" must be found, if present. Show-stoppers are insurmountable constraints, such as environmental problems (infringement on National Park treasures or endangered species, required relocation of villages, etc.) or geologic problems (tunneling through deep, extensively fractured rock, hot formation waters, noxious or explosive gases, etc.).

(2) Alternative solutions are analyzed to define the obstacles, constraints, and impacts and to determine the most feasible general scheme including preliminary project location and geometry, line and grade, as well as access locations. In the selection of line and grade, the following should be considered:

- Alternative hydraulic concepts must be analyzed, hydraulic grade lines defined, as well as the need for appurtenant structures, surge chambers, use of air cushion, etc.

- Alternatives such as shafts versus inclines and surface penstocks versus tunnels or shafts.

- Difficult geologic conditions, which may require consideration of alternate, longer alignments.

- Tunneling hazards, such as hot formation water, gaseous ground, etc.

- Tunnel depth selection to minimize the need for steel lining and to maximize tunneling in rock where final lining is not required.

- Access points and construction areas near available roads and at environmentally acceptable locations.

- Spoil sites locations.

- Schedule demands requiring tunnels to be driven from more than one adit.

- The number of private properties for which easements are required. In urban areas, alignments under public streets are desirable. Example: A long stretch of the San Diego outfall tunnel was planned to be (not actually built at this time) placed under the ocean, several hundred feet offshore, to avoid passing under a large number of private properties.

- Environmental impacts, such as traffic, noise and dust, and the effect on existing groundwater conditions.

(3) During the feasibility and early planning stages, engineering surveys must establish topographical and cultural conditions and constraints, largely based on existing mapping and air photos. Available geologic information must also be consulted, as discussed in Chapter 4, at an early time to determine if sufficient information is available to make a reliable determination of feasibility or if supplementary information must be obtained.

(4) This phase of the work should culminate in a complete implementation plan, including plans and schedules for data acquisition, design, permitting, land and easement acquisition, and construction. Strategies for public participation are also usually required.

c. Preconstruction planning and engineering.

(1) During this stage, the line and grade of the tunnel(s) and the location of all appurtenant structures should be set, and most information required for final design and construction should be obtained.

(2) Survey networks and benchmarks must be established, and detailed mapping must be carried out. Surveying required for construction control may be performed during final design. In urban areas, mapping will include all affected cultural features, including existing utilities and other facilities. Property ownerships must be researched.

(3) Geologic field mapping, geotechnical exploration and testing, and hydrologic data acquisition must also be completed in this phase and geotechnical data reports prepared. Environmental and permitting work, as well as public participation efforts, continue through preliminary design.

(4) The preliminary design will also include an assessment of methods and logistics of construction, compatible with schedule requirements. Trade-off studies may be required to determine the relative value of alternative designs (e.g., is the greater roughness of an unlined tunnel acceptable for hydraulic performance? Will the added cost of multiple headings be worth the resulting time savings?).

(5) Preconstruction planning and engineering culminates with the preparation of a General Design Memorandum, often accompanied by feature design memoranda covering separate aspects of the proposed facility.

d. The construction stage: Final design and preparation of contract documents.

(1) Contract drawings will generally include the following information:

- Survey benchmarks and controls.

- Tunnel line and grade and all geometrics.

- Site: existing conditions, existing utilities, available work areas, access, disposal areas, traffic maintenance and control, signing.

- Geotechnical data.

- Protection of existing structures.

- Erosion and siltation control; stormwater protection.

- Portal and shaft layouts.

- Initial ground support for all underground spaces, portals, shafts; usually varies with ground conditions.

- Criteria for contractor-designed temporary facilities; e.g., temporary support of excavations.

- Sequence of construction, if appropriate.

- Final lining where required (concrete, reinforced concrete, steel).

- Appurtenant structures and details.

- Cathodic protection.

- Instrumentation and monitoring layouts and details.

- Site restoration.

(2) All segments of the work that are part of the completed structure or serve a function in the completed structure must be designed fully by the design team. Components that are used by the contractor in the execution of the work but are not part of the finished work are the responsibility of the contractor to design and furnish. These include temporary structures such as shaft collars and temporary retaining walls for excavations, initial ground support in tunnels that are strictly for temporary purposes and are not counted on to assist in maintaining long-term stability, temporary ventilation facilities, and other construction equipment. When the designer deems it necessary for the safety, quality, or schedule of the work, minimum requirements or criteria for portions of this work may be specified. For example, it is common to provide minimum earth pressures for design of temporary earth retaining walls.

(3) The specifications set down in considerable detail the responsibilities of the contractor and the contractual relationship between contractor and the Government and the terms of payments to the contractor.

(4) While Standard Specifications and specifications used on past projects are useful and may serve as check lists, they are not however substitutes for careful crafting of project-specific specifications. Modern contracting practice requires full disclosure of geologic and geotechnical information, usually in the form of data reports available to the contractor. For work conducted by other authorities, a Geotechnical Design Summary Report (GDSR) or Geotechnical Baseline Report (GBR) usually is also prepared and made a part of the contract documents. This report presents the designers' interpretation of rock conditions and their effects and forms the basis for any differing site conditions claims. Preparation of such reports is not practiced by USACE at this time, with few exceptions.

e. Construction.

(1) A construction management (CM) team consisting of a resident engineer, inspectors, and supporting staff is usually established for construction oversight. This team is charged with ascertaining that the work is being built in accordance with the contract documents and measures progress for payment. Safety on the job site is the responsibility of the contractor, but the CM team must ascertain that a safety plan is prepared and enforced.

(2) During construction, the designer participates in the review of contractor submittals. Where instrumentation and monitoring programs are implemented, the designer will be responsible for interpretation of monitoring data and for recommending action on the basis of monitoring data. The design team should also be represented at the job site.

f. Commissioning and operations.

(1) Before an underground facility is declared to be completed, certain tests, such as hydrostatic testing, may be required. Manuals of operations and maintenance are prepared, and as-built drawings are furnished for future use by the operator.

(2) Permanent monitoring devices may be incorporated in the facility for operational reasons. Others may be installed to verify continued safe performance of the facility. Typical examples of permanent monitoring facilities include observation wells or piezometers to verify long-term groundwater effects.

Chapter 3
Geology Considerations

3-1. General

a. The site geology provides the setting for any underground structure. The mechanical properties of the rock describe how the geologic materials deform and fail under the forces introduced by the excavation. The geohydrologic conditions establish the quantity and pressure of water that must be controlled. Once the designer has established estimates and associated uncertainties for these parameters, the performance of the rock mass can be estimated, and the design of an underground structure can proceed.

b. The geologic stratigraphy and structure form the framework for exploring and classifying the rock mass for design and construction purposes. This geologic framework subdivides the rock mass into rock types of varying characteristics, delineates geologic boundaries, and provides clues as to geologic or hydrologic hazards. For each type of rock, intact rock properties affect stress-induced modes of behavior, durability and excavation effort, while rock mass properties—greatly affected by discontinuities and weathering—affect opening stability during and after construction.

c. This chapter describes the geologic parameters pertinent to the design of underground openings. It discusses the geomechanical properties of the intact rock and the rock mass, in situ stresses in the undisturbed rock mass, effects of weathering and discontinuities such as joints and faults on rock mass performance, and occurrences of groundwater and gases. These parameters form the basis for predicting the performance of underground structures.

3-2. Properties of Intact Rocks

a. Rocks are natural materials whose composition can be highly variable. They are usually aggregates of mineral particles although a few rocks form as amorphous glasses. Minerals are inorganic substances with unique fixed chemical compositions. The most common minerals found in rocks are given in Table 3-1. They are mainly silicates. Each mineral in a rock has physical, mechanical, and chemical properties that differ from those of other minerals present. The mineralogy of a rock is generally

Table 3-1
Common Minerals

Mineral Group	Chemical Composition	Hardness	Color	Other Characteristics
Feldspars	Aluminosilicates of potassium (orthoclase feldspar) or sodium and calcium (plagioclase feldspar) with 3-dimensional structures	6	White or grey, less commonly pink	Weathers relatively easily
Quartz	Silica, chemically very stable	7	Colorless	Breaks with conchoidal fracture
Clay Minerals	Aluminosilicates with crystal size too small to be seen with a low-powered microscope	2-3	Usually white, grey, or black	May occur as sheets that give a characteristic clayey soapy texture
Micas	Aluminosilicates of potassium (muscovite mica) or potassium-magnesium-iron (biotite mica) with sheet structures. Relatively stable minerals	2-3	Muscovite is colorless; biotite is dark green or brown to black	Break readily along close parallel planes, forming thin flakes on weathering Muscovite often twinkles in flakes on rock surface
Chlorite	Chemically a hydrous iron-magnesium aluminosilicate	2-2.5	Green	Soft, breaks readily and forms flakes
Calcite	Chemical composition $CaCO_3$	3	Ferric iron ores are red and brown; ferrous iron ores are green and grey	
Iron Ores	Oxides, Hematite (Fe_2O_3); carbonates; pyrite (FeS_2)	5-7	Dark green, brown to black	
Ferromagnesium Minerals	Chemically complex calcium and sodium aluminosilicates rich in iron and magnesium (hornblende, augite, olivine)			

Table 3-2
Moh's Scale for Measuring the Hardness of Minerals

Standard Mineral	Hardness Scale	Field Guide
Talc	1	
Gypsum	2	
		Finger nail
Calcite	3	
		Copper penny
Fluorite	4	
Apatite	5	Iron nail
	5.5	Window glass
Orthoclase feldspar	6	Penknife
Quartz	7	Steel file
Topaz	8	
Corundum	9	
Diamond	10	

determined by examination of thin sections in microscope. However, the Moh's scale of hardness (Table 3-2) provides a field procedure that can assist in identifying minerals according to their hardness and in characterizing rocks.

b. Mineral characteristics influence the engineering properties of a rock, especially when the mineral forms a significant part of the rock. Anhydrous silicates (feldspars, quartz, hornblende, augite, olivine) are considerably harder and stronger than most other common minerals and can affect the strength of a rock, its cuttability, and how it deforms. Large amounts of a relatively soft mineral such as mica or calcite can result in rapid breakdown due to weathering processes. Minerals with marked cleavage can cause anisotropy in a rock. However, since individual mineral particles are small, each particle usually has little direct influence on the mechanical properties of the rock as a whole. Although the mineralogy of a rock will influence the behavior of a rock, mechanical tests on rock samples are generally needed to define the engineering properties of rocks.

c. Rocks are broadly classified into three major groups based on their mode of origin:

(1) *Igneous rocks.* These form from the solidification of molten material that originates in or below the earth's crust. The composition depends on the kind of molten material (magma) from which it crystallizes, and its texture depends on the rate at which the material cools. Slow rates of cooling promote larger crystal-sized rock (pegmatite), whereas fast-cooling rates produce fine crystallized rock (basalt, rhyolite), or even amorphous glasses (obsidian).

(2) *Sedimentary rocks.* These form from cemented aggregates of transported fragments of rock (sandstone, siltstone, mudstone); from the accumulation of organic debris such as shell fragments and dead plants (limestone, coal); or minerals that are chemically precipitated (rock salt, gypsum, limestone).

(3) *Metamorphic rocks.* These form deep in the earth from preexisting rocks of all types in response to increases in temperature or pressure or both (gneiss, schist, slate, marble, quartzite). The composition of the metamorphosed rock depends on the original material and the temperature and pressure; its texture reflects the deformational forces.

d. Within each of these groups, separate classification systems have been developed in terms of mineral composition, grain size, and texture. The systems used for the study of geology are rather elaborate for engineering purposes, and simplifications are in order for engineering applications. Clayton, Simons, and Matthews (1982) proposed a simplified system for rock identification based on origin and grain size for igneous, sedimentary, and metamorphic rocks that provides a useful framework, within which the engineer can work. Their classification scheme for igneous rocks is given in Table 3-3 and is based on crystal size. Because crystal size is dependent on rate of cooling, the rock formation's mode of origin can be determined. The classification scheme for sedimentary rocks is given in Table 3-4. This classification is based on the mode of deposition and the chemical composition of the rocks as well as particle size. The classification scheme for metamorphic rocks is given in Table 3-5. It is based on grain structure and mineralogy.

e. Intact rock material contains grains and intergranular pores filled with air and water. The relative volumes and weights of these three constituents determine porosity, density, and saturation. The porosity of the rock has an important effect on the permeability and strength of the rock material. Other factors, such as the chemical compositions of the grains and cementation, will affect how easily it weathers or disintegrates on exposure and how abrasive it will be to cutting tools during excavation. For example,

Table 3-3
Igneous Rocks

Grain Size	Acid Light-Colored Rocks	Intermediate Light/Dark-Colored Rocks	Basic Dark-Colored Rocks	Ultrabasic Dark-Colored Rocks
Very coarse grained 60 mm	Rock consists of very large and often well-developed crystals of quartz, feldspar mica, and frequently rare minerals **PEGMATITE**			
Coarse grained	At least 50% of the rock is coarse grained enough to allow individual minerals to be identified.			Rock is coarse grained and dark in color (dull green to black) with a granular texture. It contains olivine and augite in abundance but no feldspars **PERIDOTITE**
2 mm	Rock is light colored with an equigranular texture (majority of grains approximately the same size) and contains > 20% quartz with feldspar in abundance. **GRANITE**	Rock may be medium to dark in color with more or less equigranular texture and contains < 20% quartz with feldspar and horneblende in abundance. **DIORITE**	Rock is dark colored and often greenish with abundant plagioclase (about 60%) and augite together with some olivine. The rock usually feels dense. **GABBRO**	
Medium grained	At least 50% of the rock is medium grained. Crystal outlines are generally visible with the aid of a hand lens, but individual minerals may be difficult to identify.			Rock is greyish green to black with a splintery fracture when broken and generally feels soapy or waxy to the touch. It is often crisscrossed by veins of fibrous minerals and/or banded. **SERPENTINITE**
0.06 mm	Rock is similar in appearance to granite, but the crystals are generally much smaller. **MICRO-GRANITE**	Rock is similar in appearance to diorite, but crystals are generally much smaller. **MICRO-DIORITE**	Rock is similar in appearance and often greenish with a granular texture. Individual minerals may be difficult to identify. The rock usually feels dense. **DOLERITE**	
Fine grained	At least 50% of the rock is fine grained. Outlines of crystals are not usually visible even with the aid of a hand lens. All rocks in this category may be vesicular.			
	Rock is light colored (often pale reddish brown or pinkish grey) and may be banded. **RHYOLITE** Rock is light colored with a very low specific gravity and highly vesicular. **PUMICE**	Rock is medium to dark in color (shades of grey, purple, brown, or green) and frequently porphyritic. **ANDESITE**	Rock is black when fresh and becomes red or green when weathered. The rock is often vesicular and/or amygdaloidal. **BASALT**	
Glassy	Rock is glassy and contains few or no phenocrysts. It is often black in color and has a characteristic vitreous luster and conchoidal fracture. **OBSIDIAN** Rock is glassy and contains few or no phenocrysts. It may be black, brown, or grey in color with a characteristic dull or waxy luster. **PITCHSTONE**			

clay-bearing rocks (shales and mudstones) can swell or disintegrate (slake) when exposed to atmospheric wetting and drying cycles. Typical geotechnical parameters of intact rock are shown in Table 3-6.

f. The engineering properties of a rock generally depend not only on the matrix structure formed by the minerals but also imperfections in the structure such as voids (pore space), cracks, inclusions, grain boundaries, and weak particles. Pore spaces are largely made up of continuous irregular capillary cracks separating the mineral grains. In the case of igneous rocks, a slow-cooling magma will make a relatively nonporous rock, whereas a rapidly cooling lava particularly associated with escaping gases will yield a porous rock. In sedimentary rocks,

Table 3-4
Sedimentary Rocks

Group	Detrital Sediments Bedded		Pyroclastic Sediments	Chemical and Organic Sediments Bedded	Massive Bedded
Composition and Texture	Quartz, rock fragments, feldspar, and other minerals.	At least 50% of rock is comprised of carbonate minerals (rocks usually react with dilute HCl).	At least 50% of grains are fine-grained volcanic material. Rocks often composed of angular mineral or igneous rock fragments in a fine-grained matrix.	Crystalline carbonate rocks depositional texture not recognizable. Fabric is nonclastic.	Depositional textures often not recognizable.
Grain Size Coarse grained 2 mm	Rock is composed of more or less rounded grains in a finer grained matrix: **CONGLOMERATE** Rock is composed of angular or subangular grains in a finer grained matrix: **BRECCIA**	**CALCI-RUDITE**	Rock is composed of: (i) Rounded grains in a fine-grained matrix: **AGGLOMERATE** (ii) Angular grains in a fine-grained matrix: **VOLCANIC BRECCIA**		Rock is crystalline, salty to taste, and may be scratched with a finger nail: **HALITE** (rock salt) Rock is crystalline and may be scratched with the finger nail. Grains turn into a chalky white substance when burned for a few minutes. **GYPSUM**
Medium grained 0.06 mm	Rock is composed of: (i) mainly mineral and rock fragments: **SANDSTONE** (ii) 95% quartz. The voids between the grains may be empty or filled with chemical cement: **QUARTZ SANDSTONE** (iii) 75% quartz and rock fragments and up to 25% feldspar (grains commonly angular). The voids may be empty or filled with chemical cement: **ARKOSE** (iv) 75% quartz and rock fragments together with 15% + fine detrital material: **ARGILLACEOUS SANDSTONE**	**CALCI-ARENITE**	Rock is composed of mainly sand-sized angular mineral and rock fragments in a fine-grained matrix: **TUFF**	Rock is crystalline and composed of calcium carbonate (>90%) - reacts violently with HCl. **LIMESTONE** Rock is crystalline and may show a yellowish coloration and/or the presence of voids - reacts mildly with cold dilute HCl. **DOLOMITIC LIMESTONE**	Rock is crystalline, colorless to white, frequently with a bluish tinge. It is harder than gypsum and has three orthogonal cleavages: **ANHYDRITE** Rock is black or brownish black and has a low specific gravity (1.8-1.9). It may have a vitreous luster and conchoidal fracture and/or breaks into pieces that are roughly cuboidal **COAL**

(Continued)

Table 3-4 (Concluded)

Group	Detrital Sediments Bedded		Pyroclastic Sediments	Chemical and Organic Sediments: Bedded	Chemical and Organic Sediments: Massive Bedded
Composition and Texture	Quartz, rock fragments, feldspar, and other minerals.	At least 50% of rock is comprised of carbonate minerals (rocks usually react with dilute HCl).	At least 50% of grains are fine-grained volcanic material. Rocks often composed of angular mineral or igneous rock fragments in a fine-grained matrix.	Crystalline carbonate rocks depositional texture not recognizable. Fabric is nonclastic.	Depositional textures often not recognizable.
Fine grained 0.002 mm Very fine grained	Rock is composed of at least 50% fine-grained particles and feels slightly rough to touch: **SILTSTONE** Rock is homogeneous and fine grained. Feels slightly rough to smooth to touch: **MUDSTONE** Rock has same appearance and feel as mudstone but reacts with dilute: **CALCAREOUS MUDSTONE** Rock is composed of at least 50% very fine-grained particles and feels smooth to the touch: **CLAYSTONE** Rock is finely laminated and or fissile. It may be fine or very fine grained: **SHALE**	**CALCI-SILTITE** **CHALK** (bioclastic) **CALCI-LUTITE**	Rock is composed of silt-sized fragments in a fine-grained matrix. Matrix and fragments may not always be distinguished in the hand specimen: **FINE-GRAINED TUFF** **VERY FINE-GRAINED TUFF**	Rock is crystalline and composed of magnesium carbonate (>90%). When small chip of rock is immersed in dilute HCl, there is no immediate reaction; but there is a slow formation of CO_2 beads on the surface of chip: **DOLOMITE**	Rock is black or various shades of gray and breaks with a characteristic conchoidal fracture affording sharp cutting edges. The rock cannot be scratched with a penknife: **FLINT** Rock has a similar appearance and hardness as flint but breaks with a more or less flat fracture: **CHERT**

Table 3-5
Metamorphic Rocks

Fabric Grain Size	Foliated	Massive
	Rock appears to be a complex intermix of metamorphic schists and gneisses and granular igneous rock. Foliations tend to be irregular and best seen in field exposure: **MIGMATITE**	Rock contains randomly oriented mineral grains. (Fine to coarse grained. Foliation, if present is essentially a product of thermal metamorphism associated with igneous intrusions and is generally stronger than the parent rock: **HORNFELS**
	Rock contains abundant quartz and/or feldspar. Often the rock consists of alternating layers of light-colored quartz and/or feldspar with layers of dark-colored biotite and hornblende. Foliation is often best seen in field exposures: **GNEISS**	Rock contains more than 50-percent calcite (reacts violently with dilute HCl), is generally light in color with a granular texture: **MARBLE**
Coarse grained	Rock consists mainly of large platy crystals of mica showing a distinct subparallel or parallel preferred orientation. Foliation is well developed and often nodulose: **SCHIST**	If the major constituent is dolomite instead of calcite (dolomite does not react immediately with dilute HCl), then the rock is termed: **DOLOMITIC MARBLE**
2 mm		
Medium grained	Rock consists of medium- to fine-grained platy, prismatic or needlelike minerals with a preferred orientation. Foliation is slightly nodulose due to isolated larger crystals that give rise to spotted appearance: **PHYLLITE**	Rock is medium to coarse grained with a granular texture and is often banded. This rock type is associated with regional metamorphism: **GRANULITE**
0.06 mm		
Fine grained	Rock consists of very fine grains (individual grains cannot be recognized in hand specimen) with a preferred orientation such that the rock splits easily into thin plates: **SLATE**	Rock consists mainly of quartz (95 percent) grains that are generally randomly oriented giving rise to a granular texture: **QUARTZITE (META-QUARTZITE)**

porosity will depend largely on the amount of cementing materials present and the size of grading and packing of the granular constituents. Ultimate strength of the rock will depend on the strength of the matrix and the contact between the grains.

3-3. Faults, Joints, and Bedding Planes

a. Physical discontinuities are present in all rock masses. They occur as a result of geological activities. Rock masses and their component discontinuities can be described by the following principal methods:

- Outcrop description.
- Drill core and drill hole description.
- Terrestrial photogrammetry.

b. Table 3-6 provides descriptions of the most commonly encountered discontinuities. The discontinuities introduce defects into the rock mass that alter the properties of the rock material. The mechanical breaks in the rock have zero or low tensile strengths, increase rock deformability, and provide more or less tortuous pathways for water to flow. Unless rock properties are established at a scale that includes representative samples of these defects within the test specimen, the results are not representative of the in situ rock. Therefore, parameters derived from

Table 3-6
Classification of Discontinuities for Particular Rock Types

Rock or Soil Type	Discontinuity Type	Physical Characteristics	Geotechnical Aspects	Comments
Sedimentary	Bedding planes/ bedding plane joints	Parallel to original deposition surface and making a hiatus in deposition. Usually almost horizontal in unfolded rocks.	Often flat and persistent over tens or hundreds of meters. May mark changes in lithology, strength, and permeability. Commonly close, tight, with considerable cohesion. May become open due to weathering and unloading.	Geological mappable and, therefore, may be extrapolated providing structure understood. Other sedimentary features such as ripple marks and mud-cracks may aid interpretation and affect shear strength.
	Slaty cleavage	Close parallel discontinuities formed in mudstones during diagenesis and resulting in fissility.		
	Random fissures	Common in recent sediments probably due to shrinkage and minor shearing during consolidation. Not extensive but important mass feature.	Controlling influence for strength and permeability for many clays.	Best described in terms of frequency.
Igneous	Cooling joints	Systematic sets of hexagonal joints perpendicular to cooling surfaces are common in lavas and sills. Larger intrusions typified by doming joints and cross joint.	Columnar joints have regular pattern so are easily dealt with. Other joints often widely spaced with variable orientation and nature.	Either entirely predictable or fairly random.
Metamorphic	Slaty cleavage	Closely spaced, parallel, and persistent planar integral discontinuities in fine-grained strong rock.	High cohesion where intact but readily opened to weathering or unloading. Low roughness.	Less mappable than slaty cleavage but general trends recognizable.
Applicable to all rocks	Tectonic joints	Persistent fractures resulting from tectonic stresses. Joints often occur as related groups or "sets." Joint systems of conjugate sets may be explained in terms of regional stress field.	Tectonic joints are classified as "shear" or "tensile" according to probable origin. Shear joints are often less rough that tensile joints. Joints may die out laterally resulting in impersistence and high strength.	May only be extrapolated confidently where systematic and where geological origin is understood.
	Faults	Fractures along which displacement has occurred. Any scale from millimeters to hundreds of kilometers. Often associated with zones of sheared rock.	Often low shear strength particularly where slickensided or containing gouge. May be associated with high groundwater flow or act as barriers to flow. Deep zones of weathering occur along faults. Recent faults may be seismically active.	Mappable, especially where rocks either side can be matched. Major faults often recognized as photo lineations due to localized erosion.
	Sheeting joints	Rough, often widely spaced fractures; parallel to the ground surface; formed under tension as a result of unloading.	May be persistent over tens of meters. Commonly adverse (parallel to slopes). Weathering concentrated along them in otherwise good quality rock.	Readily identified due to individuality and relationship with topography.
	Lithological boundaries	Boundaries between different rock types. May be of any angle, shape, and complexity according to geological history.	Often mark distinct changes in engineering properties such as strength, permeability, and degree and style of jointing. Commonly form barriers to groundwater flow.	Mappable allowing interpolation and extrapolation providing the geological history is understood.

Note: From A. A. Afrouz, 1992, *Practical Handbook of Rock Mass Classification Systems and Modes of Ground Failure.*

laboratory testing of intact specimens must be used with care for engineering applications.

c. The mechanical behavior of intensely fractured rock can sometimes be approximated to that of a soil. At the other extreme, where the rock is massive and the fractures confined, the rock can be considered as a continuous medium. More often, rock must be regarded as a discontinuum. The mechanical properties of discontinuities are therefore of considerable relevance. Roughness, tightness, and filling can control the shear strength and deformability of fractures. Even a tight weathered layer in a joint can considerably reduce the strength afforded by tightly interlocking roughness asperities. Discontinuities that persist smoothly and without interruption over extensive areas offer considerably less resistance to shearing than discontinuities of irregular and interrupted patterns. The orientation of fractures relative to the exposed rock surface is also critical in determining rock mass stability. Fracture spacing is important since it determines the size of rock blocks.

d. The International Society of Rock Mechanics (ISRM) Commission on Testing Methods has defined 10 parameters to characterize the discontinuities and allow their engineering attributes to be established. These are as follows:

(1) *Orientation.* Attitude of discontinuity in space. The plane of the discontinuity is defined by the dip direction (azimuth) and dip of the line of steepest declination in the plane of the discontinuity.

(2) *Spacing.* Perpendicular distance between adjacent discontinuities. This normally refers to the mean or modal spacing of a set of joints.

(3) *Persistence.* Discontinuity trace length as observed in an exposure. This may give a crude measure of the areal extent or penetration length of a discontinuity.

(4) *Roughness.* Inherent surface roughness and waviness relative to the mean plane of a discontinuity. Both roughness and waviness contribute to the shear strength. Large waviness may also alter the dip locally.

(5) *Wall strength.* Equivalent compression strength of the adjacent rock walls of a discontinuity. This strength may be lower than the rock block strength due to weathering or alteration of the walls. This may be an important component of the shear strength if rock walls are in contact.

(6) *Aperture.* Perpendicular distance between adjacent walls of a discontinuity in which the intervening space is air or water filled.

(7) *Filling.* Material that separates the adjacent rock walls of a discontinuity and that is usually weaker than the parent rock. Typical filling materials are sand, clay, breccia, gouge, and mylonite. Filling may also be thin mineral coatings that heal discontinuities, e.g., quartz and calcite veins.

(8) *Seepage.* Water flow and free moisture visible in individual discontinuities or in the rock mass as a whole.

(9) *Number of sets.* The number of joint sets comprising the intersecting joint system. The rock mass may be further divided by individual discontinuities.

(10) *Block size.* Rock block dimensions resulting from the mutual orientation of intersecting joint sets and resulting from the spacing of the individual sets. Individual discontinuities may further influence the block size and shape.

e. The ISRM has suggested quantitative measures for describing discontinuities (ISRM 1981). It provides standard descriptions for factors such as persistence, roughness, wall strength, aperture, filling, seepage, and block size. Where necessary, it gives suggested methods for measuring these parameters so that the discontinuity can be characterized in a manner that allows comparison.

f. Rock mass discontinuities more often than not control the behavior of the rock mass. Discontinuities can form blocks of rock that can loosen and fall onto a tunnel if not properly supported. Discontinuities in unfavorable directions can also affect the stabilities of cut slopes and portal areas.

g. For important structures, major discontinuities should be mapped and their effect on the structure analyzed. Additional ground support may be required to prevent particular blocks of rock from moving. It is sometimes appropriate to reorient an important structure, such as a powerhouse or a major cut, so as to minimize the effect of discontinuities.

h. It is usually not possible to discover all important discontinuities. Mapping of outcrops and oriented coring can be used to obtain statistical descriptions of joint patterns for analysis. Outcrops and cores can also be used to

obtain fracture frequencies (number of fractures per meter or foot) or average spacings. The ratio between fracture spacing and tunnel dimension or room span indicates whether the rock mass will behave more like a continuum or a discontinuum.

i. The most common measure of the intensity of rock mass discontinuities is the Rock Quality Designation (RQD), defined as the core recovery using NX core, counting only sound pieces of core longer than 100 mm (4 in.) (see Chapter 4). The RQD measure is employed to evaluate tunnel and slope stability, to estimate ground support requirements empirically, and to furnish correlations between intact rock and rock mass strength and deformation modulus.

3-4. Weathering

a. Exposed rock will deteriorate with time when exposed to the weather. The elements most critical to the weathering process are temperature and water, including water seeping through the ground. The weathering process involves both physical disintegration—the mechanical breakdown of rock into progressively smaller pieces—and chemical decomposition, resulting from alteration and replacement of the original mineral assemblage with more geochemically stable minerals, such as clay minerals and grains of quartz.

b. Freeze-thaw cycles are important physical disintegration mechanisms, occurring in many climatic environments. Diurnal and annual temperature changes also play a role. Fractures and bedding planes in the rock mass are weakness planes where there is easy access for water, naturally occurring acids, plant roots, and microbes. Therefore, the weathering process is greatly accelerated along discontinuities. As an example, limestone in a wet environment will dissolve by the action of carbonic acid and can form deep crevasses filled with weathering products or underground caverns, following the trend of faults and joints. Clay-filled joints with altered joint walls can be found at great depth where moving groundwater has had access.

c. In some environments the weathering products are or have been removed by erosional processes such as slides or streamflow. Glacial action can sweep the bedrock surface clean of weathering products and leave sound rock behind. Where weathering products remain in place, saprolite and residual soil will form. The saprolite retains many physical characteristics of the parent rock, including the texture, interparticle cohesion, and relic seams and joints. The behavior of such material can be intermediate between soil and rock. Clay infilling of cracks and joints in saprolite is often slickensided and has a low resistance to sliding, especially when wet.

d. The weathering profile is typically very irregular, because the discontinuities favor deep weathering as opposed to the solid, intact blocks. As a result, the top of weathered and sound rock below a saprolite will vary greatly in elevation, and boulders of partly weathered or nearly sound rock will be found within the saprolite.

e. The characteristics of the weathered zone is dependent on the parent rock, but even more dependent on the climate. Wet tropical climates favor deep weathering profiles; moderately wet, temperate climates in high-relief terrains favor the development of steep slopes of fresh rock, alluvial deposits, and talus. This interplay between weathering, mineralogy, and geomorphology makes it difficult to predict weathering products and profiles. Where these features and the elevation of the top of sound rock are important for an underground project, experienced geologists should provide an interpretation of the impact of these characteristics on the tunnel design.

3-5. Geohydrology

Almost all underground structures have to deal with groundwater. Water inflow during construction must be accommodated, and permanent structures may have to be made nominally watertight or designed for controlled drainage. When met unexpectedly, massive groundwater inflow can have a severe impact on construction and may require extraordinary measures for the permanent structure. It is, therefore, important to predict the occurrence and extent of groundwater and assess the effect of groundwater on the underground structure as part of site explorations. Methods of exploring the groundwater regime are discussed in Chapter 4, but methods of inflow analysis are presented in Section 3-5.e. This section gives a brief description of geologic and geohydrologic features of particular interest for tunneling.

a. Occurrence of groundwater. Groundwater is found almost everywhere below the ground surface. The hydrologic cycle includes evaporation of surface water, transport by the winds, and precipitation. Some water falling on the ground runs off in creeks and rivers, some evaporates directly or through the pores of plants, and some infiltrates and becomes a part of the body of groundwater. A tunnel or shaft will act as a sink or well unless made essentially watertight. Such an opening will disturb the groundwater regime, accept groundwater inflow, and gradually draw down the groundwater table or reduce porewater pressures

in the surrounding aquifer until a new equilibrium is obtained where inflow into the opening matches recharge at the periphery of the zone of influence. In the process, groundwater flows are often reversed from their natural directions, and aquifer release areas may become recharge areas.

b. Important geologic factors and features.

(1) For a tunnel, what is most important during construction is the instantaneous water inflow at any given location and the reduction of inflow with time. For the finished structure, the long-term inflow rates, as well as groundwater pressures around the structure, are important. The geologic features controlling these effects can be summarized as follows:

(a) The permeability of the rock mass (aquifer, water-bearing seam, shatter zone) controls the rate of flow at a given head or gradient.

(b) The head of water above the tunnel controls the initial flow gradient; the head may diminish with time. The head of water may also control external water pressures on the finished structure.

(c) The reservoir of water available to flow into the tunnel controls the duration of water inflow or the decrease of inflow with time.

(d) For the steady-state condition, groundwater recharge controls long-term water inflows.

(e) Groundwater barriers are aquitards or aquicludes of low permeability and may isolate bodies of groundwater and affect the volumes of water reservoir.

(2) Porous flow occurs in geologic materials with connected pores and where joints or other discontinuities are closed, or widely spaced, so that they do not control the flow. Examples include most unconsolidated sediments (silts, sands, gravels) and many sedimentary rocks (siltstone, sandstones, conglomerates, and other porous rocks with few or closed discontinuities). The permeability of such materials can be estimated with reasonable accuracy by packer tests in boreholes. Characterization of unconsolidated materials is often carried out using large-scale pumping tests with observation wells to measure drawdown as a function of pumping rates.

(3) Fracture flow dominates in geologic materials with low intact-rock permeability and porosity, most igneous and metamorphic rocks, and sedimentary rocks including shales, limestones, and dolomites. Fracture flow is extremely difficult to classify, characterize, and predict due to the innate variability of fractures in nature.

(4) Flow through an open fracture can be calculated theoretically, assuming parallel faces of the fracture. The flow would increase, for the same gradient, with the cube of fracture aperture. Real joints have widely varying apertures, however, and are usually partly closed, and the bulk of the flow follows intricate channels of least resistance. This phenomenon is called flow channeling. It is estimated that, in a typical case, 80 percent of the fractures do not contribute significantly to the flow, and 90 percent of the flow channels through about 5 percent of the fractures. The distribution of fracture apertures measured in the field is often highly skewed or log-normal—with small apertures dominating—yet most of the flow is through the high-aperture fringe of the distribution. It is, therefore, considered that even extensive fracture mapping (on exposures or in boreholes) will not facilitate an accurate prediction of water inflows into underground openings.

(5) Direct measurement of water flows under a gradient in a packer test is a more reliable means to characterize hydrologic characteristics of a fractured rock mass. Such tests result in equivalent values of permeability, combining effects of all fractures exposed. Even for these types of tests, however, the likelihood of intercepting the small percentage of fractures that will carry most of the flow is small, and a large number of tests are required to obtain adequate statistical coverage.

(6) When fractures are widely spaced relative to the size of the underground opening, significant water flow will occur through individual fractures. This type of inflow is highly unpredictable. On the other hand, the amount of water stored in an individual fracture is small, and flow will decrease rapidly with time unless the fracture receives recharge at close range.

(7) With more closely spaced fractures (5 to 50 fractures across the opening), a few fractures are still likely to dominate the water flow, and the inflow may be predicted, however inaccurately, on the basis of a sufficient number of packer tests.

c. Hydrologic characteristics of some geologic environments. It is beyond the scope of this manual to describe all aspects of the hydrology of geologic media. This section describes a brief selection of geologic environments, with emphasis on consolidated (rock-like) materials rather than on unconsolidated aquifers.

(1) *Igneous and Metamorphic Rocks.*

(a) These rocks almost always have low porosity and permeability, and water occurs and flows through fractures in the rock. These rock types include, among others, granite, gneiss, schist and mica schist, quartzite, slate, and some ores. Some porous flow can occur in highly altered rock in weathering zones.

(b) As a rule, the aperture of joint and fracture openings and the number of fractures or joints decrease with depth below ground due to the increase of compressive stresses with depth. However, because of the typically great strength of most of these rocks and their resistance to creep, fractures and faults can bridge and stand open even at great depth. High-water inflows have been seen in mines and in power tunnels and other tunnels many hundred meters deep (see Box 3-1).

(2) *Sedimentary rocks (consolidated).*

(a) These include conglomerates, sandstones, siltstones, shales, mudstones, marls, and others. Most of these rock types can have a high porosity (10-20 percent), but only the coarser grained of these (conglomerate, sandstone, some siltstones) have an appreciable permeability in the intact state. Thus, the coarser rocks can experience porous flow or fracture flow, or both, depending on the character of fracturing. Flow through the finer grained sediments, however, is essentially fracture flow.

(b) Fractures in the softer sedimentary rocks are more likely to close with depth than in the igneous and metamorphic rocks. In layered sediments, many joints are short and do not contribute much to water flow. Joints are often particularly numerous in synclines and anticlines as compared with the flanks of folds.

(3) *Volcanic rocks.*

(a) Basalts and rhyolites are often laced with numerous fractures due to cooling during the genesis of these rocks. Most of the water from these formations, however, comes from ancillary features. Plateau basalts are formed in layers with vesicular and brecciated material on top of each layer. Sometimes interlayer weathering and deposition is found. Hawaiian basalt typically follows sequences of pahoehoe, lava, and clinkers. Some of the interlayers can carry immense amounts of water.

(b) Basalt flows also feature large tubes created when liquid lava emptied out from under already hardened lava, as well as other voids such as those left behind trees inundated by the lava flow.

(c) Formations such as welded tuff can be highly vesicular and porous, and contain numerous cooling fractures. Thus, both porous and fracture flow can occur.

(4) *Effects of faults and dikes.*

Box 3-1. Case History: San Jacinto Tunnel, California

The San Jacinto water tunnel was completed in 1939 for The Metropolitan Water District of Southern California as part of the Colorado Aqueduct project. The 6-m-diam, 21-km-long tunnel was excavated through mostly granitic rocks with zones of metamorphic rock (mica schist, quartzite, marble) at an average depth of about 450 m. Four major faults and about 20 minor faults or fractures were encountered. There were 8 or 10 instances when peak flows of 1,000-1,100 l/s (15,000 gpm) were experienced, with estimated maximum pressures of up to 4.2 MPa (600 psi) but more commonly at 1-2.5 MPa (150-350 psi).

The large surges of inflow usually occurred when tunneling through impermeable major fault zones, notably the Goetz Fault, which held back compartments of groundwater under high head. Another fault, the McInnes Fault, was approached by tunneling from both sides. Drainage into the Goetz Fault and other faults had depleted the reservoir. This resulted in an inflow less than 6 l/s (100 gpm) when the McInnes Fault was crossed.

It was estimated that the tunnel job had depleted some 155,000 acre-feet of water from the aquifers; springs were affected at a distance of 5 km (3 mi).

Source: The Metropolitan Historical Record, 1940.

(a) Small faults are often the source of fracture flow into tunnels. Larger faults or shear zones have been known to produce water inflow of the order of 3,600 l/s (50,000 gpm). The permeability of the geologic material in a shear zone can be highly variable, depending on whether the zone contains mostly shattered and sheared rock or large quantities of less permeable clay gouge or secondary depositions. In many cases, faults act as a barrier between two hydrologic regions. This happens when the fault zone material is less permeable than the adjacent, relatively permeable geologic material, or when a fault offsets less permeable strata against aquifers. Thus, for one reason or another, formation water pressures can be much higher on one side of a fault than the other. Tunneling through a fault from the low-pressure side can result in sudden and unexpected inflow of water.

(b) Many geologic environments are laced with dikes. The original formation of the dikes often disturbed and fractured the host material, and locally the permeability can be many times larger than the main body of the rock mass. On the other hand, the dike material, if not badly fractured, can be tight and form a groundwater barrier much like many faults. Examples of dikes acting as water barriers abound in Hawaii, where dikes crossing very pervious clinker layers can form adjacent compartments with widely differing groundwater levels.

(5) *Interface between rock and overburden.* Since bedrock is usually less pervious than the overburden, perched water is often found above bedrock. Coarse sediments are often found just above bedrock. Even cohesive residual soils above bedrock are often fractured and contain water. It is therefore important to pay attention to the bedrock interface, because it can cause difficulty in construction of shafts and inclines, as well as for mixed-face tunneling. In cold climates, seepage water will form ice and icicles, which can be hazardous when falling, especially into shafts.

(6) *Rocks subject to dissolution.*

(a) These include limestone, gypsum, anhydrite, halite and potash, and rocks cemented with or containing quantities of these types of materials.

(b) Calcite is only mildly soluble in pure water, but meteoric water contains carbon dioxide from the air, which forms carbonic acid in the water, able to dissolve calcite. Thus, water flowing through fractures in limestone over time can remove portions of the calcite, leaving open fissures or cavities, even caves behind. Larger cavities tend to form where joints or faults intersect. If near the surface, such dissolution can eventually result in sinkholes. Karstic landscapes are limestone regions with advanced dissolution, where pinnacles of limestone remain and where essentially all water flow is through underground caverns rather than in rivers on the surface. Examples are found in Kentucky, Puerto Rico, and Slovenia. Clearly, tunneling through limestone with water-filled cavities can be difficult or even hazardous. On the other hand, limestones that have never been subject to dissolution can be most ideal for tunneling, being easy to excavate yet self-supporting for a long time.

(c) Formation water often contains much more carbon dioxide than meteoric water and is thus able to contain more calcite in solution. This carbon dioxide comes from sources other than rain infiltration, such as oxidation of underground organic materials. If formation water containing excess carbon dioxide is released to the atmosphere at normal pressure, carbon dioxide is released from the water to form a new equilibrium with carbon dioxide in the air. Hence, calcite is precipitated as a sludge that can harden when exposed to air. This occasionally results in a clogging problem for tunnels and other underground works that incorporate permanent drainage systems.

(d) Underground works for USACE projects rarely encounter halite or other evaporites. These are most often exposed in salt or potash mines or, for example, in nuclear waste repository work such as the Waste Isolation Pilot Project in New Mexico. If drainage occurs into underground works in or near such geologic materials, rapid dissolution can result, causing cavities behind tunnel linings and elsewhere and instability of underground openings. Shafts through or into these materials must be carefully sealed to prevent water inflow or contamination of groundwater.

(e) Some geologic materials are cemented by soluble materials such as calcite or gypsum, existing either as interstitial cement or as joint fillings. Gypsum is dissolved rapidly by moving formation water, while calcite is dissolved more slowly. The San Franciscito Dam in southern California failed largely because groundwater flow resulting from the impoundment of water behind the dam dissolved gypsum cement in the rocks forming the abutments of the dam. In such geologic materials, underground structures should be made watertight.

(7) *Thermal water.* Hot springs occur at numerous locations in the United States, in all of the states from the Rocky Mountains and westward, in the Ozarks in Arkansas, and in a narrow region along the border of Virginia and West Virginia. The source of the hot water is either

meteoric water that finds its way to deep, hot strata, or the water is magmatic, or a mixture of the two. The hot water finds its way to the ground surface, helped in part by buoyancy, through preferred pathways such as faults or fault intersections. The hot water often contains minerals in solution. Apart from the obvious problems of dealing with large quantities of hot water underground, the water is also difficult to dispose of in an environmentally acceptable fashion.

d. Analysis of groundwater inflow.

(1) Groundwater causes more difficulty for tunneling than any other single geologic parameter. Groundwater inflow is one of the most difficult things for tunnel designers to predict, yet many decisions to be made by the designer as well as the contractor depend on reasonable assessments of groundwater occurrence, inflow, and potential effects. Inflow predictions are needed for at least the following purposes.

(a) *Leakage into or out of permanent structures.* Decisions regarding choice of lining system depend on an assessment of leakage inflow.

(b) *Groundwater control during shaft sinking.* Often the overburden and the uppermost, weathered rock will yield water that must be controlled to prevent instability, excessive inflow, or quicksand conditions. Deeper, pervious strata may also offer insurmountable problems if water inflow is not controlled. Decisions must be made concerning the control of water inflow. Water can be controlled by construction of slurry walls, grouting, freezing, installation of wells, or a combination of these methods.

(c) *Groundwater control during tunneling.* Decisions must be made regarding whether probing ahead is required in some or all reaches of the tunnel, whether dewatering or grouting in advance or from the tunnel face will be required, or perhaps whether an alternate route might be better in order to avoid high-water inflows.

(d) *Pumping requirements.* A reasonable estimate of water inflow must be made so that the contractor can acquire appropriate pumping and dewatering equipment. This is especially important when driving a tunnel downgrade or from a shaft. Water inflow also affects tunnel driving rates, whether by tunnel boring machine (TBM) or blasting.

(e) *Environmental effects.* It is often necessary to estimate the extent of water table drawdown, temporary or permanent, for reasons of environmental protection (protection of groundwater to sustain vegetation or of groundwater rights).

(2) These impacts can affect the requirements of groundwater flow analyses to estimate of the maximum expected flow rate and volume. Pump-size estimates may be the end result of groundwater inflow calculations. Conservative estimates may be appropriate for design purposes; however, overly conservative calculation may impact costs (since cost is affected by the chosen method of dealing with inflow).

(3) In contrast, where environmental issues are concerned, the needs of groundwater analysis can have a qualitatively different impact on the project. If the source of water affected by tunnel dewatering is a surface water system of environmental significance, the calculated volumes and disposal methods can affect the basic feasibility of the project. For example, increasingly stringent requirements for wetland protection can affect any project in which a significant fluctuation in the groundwater level is anticipated. If the dewatering program is calculated to produce a significant drawdown in a wetland, the precise calculation of withdrawal rates is important. The viability of a project can, in principle, rest on the ability to demonstrate that the project will not significantly affect the prevailing hydrologic regime.

(4) As a result, the designer may be faced with the need to reconcile very different requirements and to apply sophisticated techniques to obtain the necessary estimates of groundwater behavior. The methods of control can also vary, depending on the situation. Pumping or draining may not be adequate as control measures if the impact on the surrounding hydrogeologic system is to be minimized. Measures to prevent or mitigate the inflow of water to the tunnel may be required instead of pumping.

e. Modeling of groundwater flow.

(1) The basic principles that govern the choice of methods for groundwater flow estimation requires that the designer identify a conduit for flow (a fracture network or inherent permeability), a source of water (entrained in the rock or available elsewhere), and a gradient (determined by suitable boundary conditions and permeability of the rock medium). These requirements imply that the geometry of the system, the characteristics of the matrix, and the available sources of water must be identified. It is impossible to assess all of these for the reasons discussed above. Therefore, uncertainty will be associated with groundwater-flow estimation. Reducing this uncertainty to acceptable limits is a desirable objective, but generally a difficult if

not impossible one to achieve. This is because uncertainty lies not only in the physical system, but in the method of analysis.

(2) The physical system can only be approximated. Even though geotechnical and geophysical techniques can supplement direct observation to produce better estimates of the physical characteristics of the rock matrix, the present state of the art in geologic interpretation does not permit perfect knowledge of that matrix. The fracture network has a random component; permeability is a variable; and the location of connected water bodies as well as the recharge of those bodies are not perfectly quantifiable. As a result, even though extensive testing can produce reasonable estimates of the rock hydrogeology, those estimates are, at best, imperfect.

(3) Even the mathematics of groundwater flow are not perfectly known. It is usually assumed that Darcy flow applies, i.e., that flow is directly proportional to gradient. This is a reasonable approximation for water in a porous medium such as a sand. However, for media where fractures govern, the characteristics of flow often depart from the Darcy assumption.

(4) The sequence of analysis will depend on the specific problem, but should generally have the following characteristics:

(a) *Define the physical system.* Principal rock and conduit characteristics must be identified. Aquifer and aquiclude units and conduits or irregularities should be located. Since the scale of the problem affects the area of the physical system that is of interest, some approximating formulae or methods of analysis may be appropriate at this stage. Given this starting point, the extent of the physical system can be estimated, and characteristics within that extent can be defined.

(b) *Determine governing boundary conditions.* Water bodies, aquicludes, or other factors limiting the propagation of changes in the hydraulic gradient induced by tunneling must be determined. Since this step is closely related to the definition of the physical system, determination of boundary conditions should be done in concert with the definition of the rest of the physical system.

(c) *Identify characteristic hydrogeologic flow system.* The way the system behaves in terms of hydraulic flow patterns (fracture flow, permeable matrix flow, etc.) must be identified based on the known physical system, boundary conditions, and approximations of hydrogeologic parameters.

(d) *Estimate of hydrogeologic parameters.* Estimates of system geometry, permeability, source volumes, and similar factors that govern flow within the system must be made. At this point, the parameters of interest will depend on the physical system that has been defined. In a medium treated as porous, permeability will be important. In a medium with fractures that might be principal flow conduits, hydraulic conductivity may take on other meanings.

(e) *Select method of analysis.* Given the defined hydrogeologic problem, a model or models should be selected. The analysis, including model calibration, validation of model performance, generation of results, and testing of sensitivity can then proceed. A large number of commercially available and public-domain computer codes are available for two- or three-dimensional (2- or 3-D), steady-state, and transient flow analysis. Sometimes, simple closed solutions will have sufficient accuracy.

(5) An important part of the analysis process is to verify that the initial selection of model boundaries was adequate. If the simulated results indicate that artificial boundary conditions are being generated, then the extent locations of boundaries must be revisited. Indications of this are contour lines bending at the perimeter of a mathematical model or fixed boundaries generating large quantities of flow. Further checks should be made in terms of the estimates of volume loss. The processes that govern recharge in the system should be checked to verify that simulated rates of withdrawal are sustainable. If the model predicts a long-term loss rate greater than natural recharge over the extent of the system (e.g., from rainfall or other factors), then the model results must be checked.

f. Simplified methods of analysis.

(1) It is important to distinguish between different types of groundwater inflow. Depending on the character of the water source, field permeability data can be applied to flow equations for predictive purposes. The types of inflow can be classified as follows:

- Flow through porous rock.
- Flow through fractures in otherwise impervious rock.
- Flow through shatter zone, e.g., associated with a fault.
- Flow from an anomaly, such as a buried river valley, limestone cave, etc.

(2) Each of these types of flow require a different approach to arrive at reasonable groundwater inflow estimates. In most cases, however, a set of simple equations may be adequate for analysis.

(3) Flow through porous rock, such as a cemented sand or an unfractured sandstone, is reasonably regular and predictable. In such rocks, the permeability of the rock mass is a reasonably well-defined entity that can be used with confidence in analyses. The reliability of any prediction can be judged on the basis of the uniformity or variability of permeability data from field tests. In stratified materials, the permeability of the material is likely to be greater in the direction of the bedding than across the bedding. This affects not only the inflow prediction but also the borehole permeability data interpretation that is the basis of prediction.

(4) Porous rocks often have a large pore volume (10-30 percent or more) and thus contain a substantial reservoir of water that will take time to drain. In fractured rock of low porosity and permeability, water flows through the fractures, which are usually of variable aperture, have a variety of infillings, and appear in quantity and direction that can be quite random or regular, depending on the characteristics of the jointing patterns. As a result, the permeability of the rock mass is poorly defined, likely to be highly variable and scale dependent, and with unknown anisotropy; the permeability measured in the field is usually a poor representation of the actual nature of the flow of water. However, an interpretation of the data can be made in terms of equivalent permeability and geometry and used in an appropriate formula to obtain approximate results.

(5) Typically, fractured rock offers only a small storage volume. Therefore, water flows often reduce drastically in volume after a short while, unless the fractured rock aquifer has access to a larger reservoir. On rare occasions, a rock mass features porous flow and fracture flow of about equal equivalent permeability.

(6) A common occurrence is inflow through a zone of limited extent, such as a shatter zone associated with a fault, or a pervious layer in an otherwise impervious sequence of strata. With permeability measurements available and a reasonable estimate of the geometry, inflow estimates can be made using one of the equations for confined flow.

(7) Inflow from large anomalies must be judged and analyzed on a case-by-case basis. Theoretically, flow through caverns or caves can be analyzed the same way as channel flow. In practice, however, the data are not available to perform these types of analyses. In any event, the mere presence of these types of anomalies with large quantities of water will require remedial measures of one kind or another, and a precise estimate of the potential inflow is not necessary.

(8) It is a common experience that water inflow into a tunnel decreases with time from the initial burst of water to a steady-state inflow rate of only 10-30 percent of the initial inflow rate. Steady-state flow equations can be used to determine inflows based upon assumed boundary conditions. These boundary conditions will change with time, as the groundwater reservoir is depleted. It is possible to obtain a rough estimate of the decreasing rate of flow using the steady-state equations, based on estimated geometric extent and porosity of aquifer reservoir. This method will only yield order-of-magnitude accuracy. If available data warrant greater accuracy of the analysis, transient flow can be estimated using numerical analyses.

(9) A number of problems can be analyzed using the flow net method. Flow nets are graphical solutions of the differential equations of water flow through geologic media. In a flow net, the flow lines represent the paths of water flow through the medium, and the equipotential lines are lines of equal energy level or head. The solution of the differential equations require these two sets of curves to intersect at right angles, when the permeability of the medium is isotropic and homogeneous. Detailed instructions of how to draw flow nets are not presented here. Such instructions can be found in a number of textbooks. The flow net method is suitable for solving problems in 2-D, steady-state groundwater flow. Anisotropy of permeability can be dealt with using transformations, and materials of dissimilar permeability can also be modeled. The method produces images of flow paths and head and can be used to estimate flow quantities, gradients, and pressures, and to assess effects of drainage provisions and geometric options. The example shown in Figure 3-1 demonstrates its use as a means to estimate the effect of drains on groundwater pressures on a tunnel lining. The flow net is hand drawn, crude, and flawed, yet provides information of sufficient accuracy for most purposes. In addition to the flow path and head distribution, the figure shows the estimated hydrostatic pressure on the lining with drains as shown. The water flow can be estimated from the number of flow channels, n_f, and the number of potential drops, n_d, together with the total head h:

$$q = khn_f/n_d$$

g. Limitations of simplified methods of analysis.

(1) The differential equations governing groundwater flow are not inherently complex, but are of a form that do not readily lend themselves to direct solution. As a result, analytic solutions to groundwater flow problems are generally derived for special simplified cases of the general problem. These simplifications generally take the form of assuming homogeneous and/or isotropic media, tractable boundary conditions, steady-state conditions, and/or simplified source/loss terms. Literally dozens of such special case solutions exist, and they have been used in a variety of problems.

(2) Anisotropy and other complicating factors are the rule rather than the exception; therefore, simplified methods must be used with caution. The assumed range of influence in a well function, for example, is commonly seen as a characteristic of the medium and the withdrawal rate. In fact, in the long term this factor represents the distance to a boundary condition that limits the extent to which drawdown can occur. A well function drawdown equation, however, can provide a useful approximation of events under some conditions. Given the ready availability of a number of mathematical models that provide easy access to better solutions, analytic solutions have their place in analysis for tunnels and shafts in two main areas. They can be used to provide a useful order of magnitude check on model performance to verify the basic model behavior and as first-cut approximations that help in problem definition during the basic steps in analysis described above.

(3) At present, the state of the art of computer simulation using finite element or finite difference techniques has progressed to the point where these models are relatively easily and effectively applied. Although use of such models in a complex 3-D system can present a challenge, the models when properly applied can be used with confidence. Errors may result from either the uncertainty in measurement of the physical system or, as noted above, from inappropriate assumptions as to the mechanics of flow in the system. These errors are common to all of the above methods. The use of a comprehensive finite model, and not analytic solutions or flow nets, will reduce errors introduced by simplification of the physical system to a minimum.

(4) An important part of the process of analysis lies in the recognition of the basic nature of flow in fractured rock systems. If the physical system can be approximated as a continuum in which Darcy's law applies (i.e., a porous medium), such as a sand or sandstone, the problems of analysis are relatively straightforward.

(5) In a fractured medium, the fractures that dominate flow can be approximated as a continuum system with a permeability and porosity representing the net effect of the fracture system. This assumption is appropriate, provided that no single or limited number of fractures dominates and that the hydraulics of flow can be represented by an approximating medium in which the average effect of a large number of randomly placed and interconnecting fractures can be represented by an average effective hydraulic conductivity. This approach may be reasonable provided that the system is such that flow is approximately proportional to gradient, and flow is not dominated by a small number of fractures.

(6) Most difficult is the case where fractures are large and randomly placed. As observed above, in such a system the permeability of the rock mass can be overwhelmed by the conductivity of a single channel, which provides a hydraulic conduit between the source of water and the tunnel. Even if it is known for certain when such a fracture will be encountered, the hydraulics of flow can be difficult to establish. Effective conduit size, length, section, and roughness, which all have an impact on flow rate, can be highly variable.

(7) Given that the likelihood of encountering such a fracture often can only be estimated, the size of the required pumping system can be difficult to establish. If available pressure head is known, and the approximate section of a fracture can be estimated, then the hydraulics that govern the flow can be estimated by taking an equivalent hydraulic radius, section, and roughness. If these parameters are treated as random variable analysis and a statistical analysis is performed to produce a variability for each of these factors, confidence limits can be determined.

(8) Alternatively, calculating flow for a range of critical sizes and hydraulic characteristics can produce estimates of potential flow rates. The problem of solving for the likelihood of intersecting a particular number of independent fractures then arises. Treating the problem as one of a spatially distributed variable, it is possible to generate estimates of this occurrence provided that the fracture system has been sufficiently well characterized. In practice, the most likely compromise is to estimate the probable effective hydraulic characteristics of a fracture, estimate the rate of intersection (fractures per tunnel mile), and add a safety factor to the design of dewatering facilities.

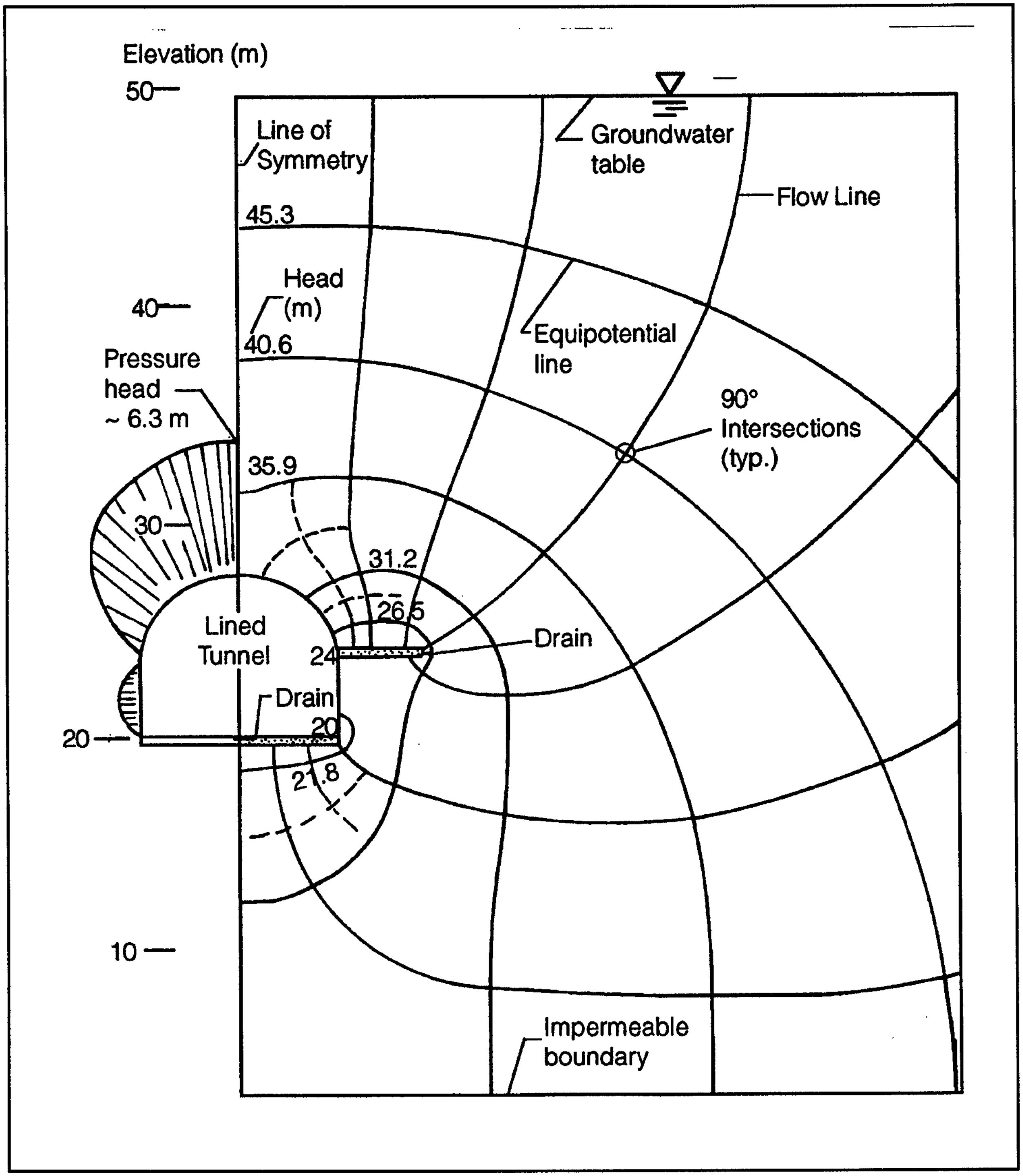


Figure 3-1. Flow net for analysis of inflow and lining pressure, tunnel in homogeneous material

3-6. Gases in the Ground

Natural gases are encountered rarely in tunneling. However, when natural gases enter tunnels and other underground openings, they pose a particularly severe hazard that can, and often has, resulted in death to workers. Gases are often found in unexpected areas and are difficult to detect, unless monitoring stations are set up for the purpose. It is necessary, therefore, to determine during design the risk of encountering gas during construction, so that appropriate measures can be taken to eliminate the hazards of gas exposure. A specific effort should be made during the explorations phase to determine the risk of encountering gas during construction and to classify the works as gassy, potentially gassy, or nongassy. This effort should include research into the history of tunneling in the particular geographic region, interpretations of the geologic and geohydrologic setting, measurement of gas content in air samples from boreholes, geophysical methods to assess the existence of gas traps in the geologic formations, and other methods as appropriate. To aid in the planning and execution of such explorations and interpretations, the following subsections describe briefly the origin and occurrence of various gases in the subsurface. Safety aspects of gas in underground works are further discussed in Section 5-11.

a. Methane gas.

(1) *General.* Of all the naturally occurring gases in the ground, methane gas is the most common and has resulted in more accidents and deaths than any other gas. In the United States, occurrences and fatal accidents in civil engineering tunnel projects have been reported, among others, in the following localities:

- Los Angeles Basin (occurrence in a number of water and rapid transit tunnels, fatal explosion in the San Fernando water tunnel at Sylmar, 1971).

- Port Huron, Michigan (accident in sewer tunnel through Antrim Shale, 1971).

- Rochester, New York (occurrence in sewer tunnel through Rochester Shale).

- Milwaukee, Wisconsin (accident in sewer tunnel through porous sandstone).

Other occurrences in tunnels include Vat, Utah; Richmond, New York; Euclid, Ohio; and Soliman, California. Methane emissions measured in these tunnels have averaged 2-25 l/s (5-50 cfm), with peak emissions up to 200 l/s (400 cfm) (Critchfield 1985).

(2) *Sources.*

(a) There are several sources in nature for the generation of methane gas. The most common origin of methane gas in large quantities is thermomechanical degradation of organic materials at great depth. This is a process related to the generation of coal, anthracite, and hydrocarbons, and methane gas is, therefore, often found in association with coal and anthracite strata and with oil fields. Coal mines are frequently affected by steady inflows and occasional outbursts of methane (coal can contain a volume of 10 m^3 of methane per m^3 of coal), and methane is a common byproduct of crude oil production. Other volatile hydrocarbons usually accompany the methane.

(b) Another source of methane gas is near-surface bacterial decay of organic matter in sediments with low-oxygen environment, such as in peats and organic clays and silts, and in marshes and swamps with stagnant water (marsh or swamp gas). This source generally produces much smaller flow rates than sources associated with coal or oil. In glaciated environments, methane is often generated in interglacial organic deposits such as interglacial peat bogs. Methane is also generated in man-made organic landfills. Methane can also result from leakage out of natural gas and sewer lines and sewage treatment plants, and abandoned wells may provide conduits for gas flows.

(c) Knowledge of the origins of methane gas and other volatile hydrocarbons is important for the assessment of the risk of encountering gas. However, the occurrence of such gases is by no means restricted to the strata of their origin. While solid carbons will remain in place in the strata of origin, liquid hydrocarbons will flow into other strata in a manner determined by gravity, geologic structure, and strata porosity and permeability. Gas will seek a path to the ground surface through permeable strata until released at the ground surface or trapped in a geologic trap that prevents its release. Thus, gas can be found many miles away from its origin in strata that have no other traces of carbon or hydrocarbon. In fact, gas has been found in rock formations ranging from pegmatite, granite, and other igneous or metamorphic rocks to shale, mudstone, sandstone, and limestone, and in mines for copper, diamonds, iron, gold, uranium, potash, or trona. Gas is also often found in salt deposits, either dissolved in brine or as gas pockets in voids. Such gas pockets in salt under pressure sometimes cause violent outbursts when mining occurs close to the gas pocket.

(d) Geologic gas traps are formed by several kinds of geologic structures. Gas traps are commonly found in association with deformed strata adjacent to salt domes, often with liquid hydrocarbons. Fault displacements sometimes juxtapose pervious and impervious layers to create a gas trap. Folded strata also form traps, especially in anticlines and monocline. Impervious clay strata in glacial sediments can form traps for gas originating from interglacial organic deposits or deeper origins.

(e) As other gases, methane often occurs in gas form in the pores, fractures, and voids of the rock mass. Breakage of the rock or coal and exposing wall surfaces liberates the gas. However, large quantities of gas can be dissolved in the groundwater. Water can contain methane and other gases in solution in concentrations that depend on the water temperature and the hydrostatic pressure in the water. When water is released into an underground opening, the pressure drops drastically, and the ability of the water to contain gases in solution virtually disappears. Hence, the gases are released into the tunnel in quantities that are proportional with the amount of water inflow.

(3) *Levels.* Methane is lighter than air (density 55 percent of air) and in stagnant air tends to collect in air traps in underground works. When mixed, however, it does not segregate or stratify. Methane is explosive in mixtures of 5 to 15 percent. In general, the methane level should be kept below 0.25 percent, and a methane content above 1 percent is usually unacceptable.

(4) *Construction.* Construction in the presence of toxic, flammable, or explosive gases is regulated by OSHA (29 CFR 1926). Guidance can also be found in MSHA (30 CFR 57). Some states have stricter rules, such as the State of California's Tunnel Safety Orders. Minimum requirements and provisions for dealing with flammable or toxic gases are presented in the California Tunnel Safety Orders, as well as in OSHA (29 CFR 1926).

(5) *Classifications.* These Safety Orders classify tunnels as follows:

(a) Nongassy classification shall be applied to tunnels where there is little likelihood of encountering gas during the construction of the tunnel.

(b) Potentially gassy classification shall be applied to tunnels where there is a possibility flammable gas or hydrocarbons will be encountered.

(c) Gassy classification shall be applied to tunnels where it is likely gas will be encountered or if a concentration of 0.25 percent by volume (5 percent of LEL [lower explosive limit]) or more of flammable gas has been detected not less than 12 in. from the roof, face, floor, and walls in any open workings with normal ventilation.

(d) Extrahazardous classification shall be applied to tunnels when the Division [of Industrial Safety] finds that there is a serious danger to the safety of employees and flammable gas or petroleum vapors emanating from the strata have been ignited in the tunnel, or a concentration of 20 percent of LEL petroleum vapors has been detected not less than 3 in. from the roof, face, floor, and walls in any open workings with normal ventilation.

b. Hydrogen sulfide.

(1) Hydrogen sulfide is lethal in very small quantities. Its characteristic smell of rotten eggs is evident even at very small concentrations (0.025 ppm), and low concentrations quickly paralyze the olfactory nerves, deadening the sense of smell. Hence, smell cannot be relied on, and the presence and concentration of hydrogen sulfide must be measured. The safety threshold limit for 8 hr of exposure is 10 ppm. Higher concentrations cause membrane irritation; concentrations over 700 ppm may not be survivable.

(2) Hydrogen sulfide is a product of decay of organic materials; it is often associated with the occurrence of natural gas and liquid hydrocarbons, but has also been found in swampy areas or near sewers, landfills, and refineries. It is highly soluble in water and is often carried into underground openings with water inflow, and is sometimes produced by reaction between acid water and pyrite or marcasite. It is also common in association with geothermal water and volcanic emissions.

(3) Hydrogen sulfide, like methane, is flammable or explosive in the range of 4.3- to 45.5-percent concentration in air.

c. Sulphur dioxide and other gases.

(1) Sulphur dioxide results from oxidation of sulphur or sulfides in sediments and in hydrothermal deposits with sulfides, or directly from volcanic action, but is encountered more commonly as a component of blast fumes, fire, and combustion engine exhaust. Sulphur dioxide is toxic with a safety threshold value of 2 ppm.

(2) Carbon dioxide derives from carbonaceous materials subject to oxidation or effects of acid water. This is an asphyxiant with a threshold level of 5,000 ppm; it is toxic above 10,000 ppm. An excess of carbon dioxide is often

associated with depletion of oxygen. Carbon dioxide is heavier than air and settles into depressions, shafts, or large drillholes for caissons or wells where asphyxiation can become a real danger. Carbon dioxide is also found in hot water from deep origins and in geologic strata.

d. Other gases.

(1) Hydrogen occurs occasionally in association with hydrocarbons and is explosive.

(2) Radon gas is a decay product of uranium. Radon and its first four decay products are hazardous because of their emission of alpha particles during their relatively short half-lives. These alpha particles can cause respiratory cancer. Radon is found in uranium mines, where the hazard is controlled by dilution with increased ventilation, sometimes supplemented by installation of membranes and rock coatings. Radon is also found in the pores and fractures of other rock types that contain uranium, especially metamorphic and igneous crystalline rocks such as gneiss and granite, but also in some shale beds. Groundwater contained in these types of formations also often carry radon in solution. The presence of radioactive materials can be detected by borehole probes. Radon detectors can detect the presence and activity of radon in borehole or tunnel air.

Chapter 4
Geotechnical Explorations for Tunnels and Shafts

4-1. General

a. Geological, geomechanical, and hydrological factors more than any other factors determine the degree of difficulty and cost of constructing an underground facility. Chapter 3 of this Manual discusses many of the geological factors that affect underground works. This chapter presents guidelines for acquiring the necessary geological data for the planning, design, and construction of underground works.

b. In brief, the types of information that must be obtained can be classified as follows:

- Geologic profile (stratigraphy, structure, and identification of principal rock types and their general characteristics).
- Rock mass characteristics and geomechanical properties.
- Hydrogeology (groundwater reservoirs, aquifers, and pressures).
- Exposure to construction risk (major water-bearing faults, methane gas, etc.).

c. USACE's Engineer Manual 1110-1-1804, Geotechnical Investigations, and EM 1110-1-1802, Geophysical Exploration, contain information useful for the planning and execution of geotechnical explorations for tunnels and shafts.

4-2. Explorations for Reconnaissance and Feasibility Studies

a. General. The project is conceived, defined, and broadly scoped out during the reconnaissance phase. Geotechnical information required during this phase is obtained almost exclusively from existing data, with a minimum of field work. More information is required to conduct feasibility studies. Here the emphasis is first on defining the regional geology and the basic issues of design and construction. Methods of data acquisition include at least the following:

- Available data acquisition and study.
- Remote sensing.
- Preliminary geologic field mapping.
- Geophysical explorations if appropriate.
- Selected exploratory borings in critical locations.

b. Sources of available information.

(1) Topographic maps are available for every location in the United States. They are useful in showing geologic domains and often, by interpretation, show geologic structures. Geologic maps are also available for virtually every location in the United States. These may be obtained from the U.S. Geologic Service (USGS), state geologic services, university publications, or private sources such as mining companies. Some private information is proprietary and may not be available for use.

(2) In urban areas and where site improvements have been made (e.g., highways), private and public owners will frequently have information about past geotechnical and geologic investigations. Local geotechnical firms regularly maintain files of such information.

(3) Much of the available information will have been collected for purposes other than engineering evaluations (e.g., resource assessments), and interpretive work is required of the engineering geologist to extract the information that is useful for tunnel and shaft design and construction. The end product of office studies is a set of geologic maps and profiles, descriptions of rock types, and a list of potential difficulties, all subject to field verification or verification by other means.

(4) Case histories of underground works in the region, or in similar types of rock, are sometimes available and are very useful additions to the geotechnical database.

(5) The collection and analysis of available data must also include geographical, cultural, and environmental data, such as land ownership, existing facilities, access routes, environmental sensitivity, etc. Local resource developments, such as quarries, mines, or oil wells, should also be mapped.

c. Remote sensing techniques.

(1) Every location in the United States has been photographed from the air at least once and many locations numerous times, and most of these air photos are available at a low cost, from private or public sources. The typical

black-and-white stereo coverage usually used for topographic mapping is very well suited for geologic interpretation and will divulge details such as landform definition, boundaries between rock and soil types, lineaments, landslides, drainage features, archaeological sites, etc. Color photos are useful for land use determination. False-color photos are used for special purposes. Infrared photos show temperature differences and are useful, for example, in defining moisture content contrasts of the ground, as well as drainage paths.

(2) In built-up areas, air photos cannot show much of the natural ground, but it is often possible to find older photos from the time before construction. A series of older sets of photos sometimes are handy for tracing the past history of a locality. Satellite coverage is now available from public sources (and some private sources) in many forms and to many scales, and made for many purposes. Aerial photography is used to supplement existing mapping data and to identify additional geologic features, useful for field verification and for planning additional site exploration work. Air photos are also useful for overlaying alignment drawings.

d. Field mapping.

(1) Initial onsite studies should start with a careful reconnaissance over the tunnel alignment, paying particular attention to the potential portal and shaft locations. Features identified on maps and air photos should be verified. Rock outcrops, often exposed in road cuts, provide a source for information about rock mass fracturing and bedding and the location of rock type boundaries, faults, dikes, and other geologic features. In particular, the field survey should pay attention to features that could signify difficulties:

- Slides, new or old, particularly in portal areas.
- Major faults.
- Sinkholes and karstic terrain.
- Hot springs.
- Volcanic activity.
- Anhydrite, gypsum, or swelling shales.
- Caves.
- Stress relief cracks.
- Zones of deep weathering or talus.

(2) Once alignment and portal site alternatives have been set, a detailed geologic mapping effort should be carried out. Joints, faults, and bedding planes should be mapped and their orientations plotted by stereographic projection so that statistical analysis can be performed (often, today, with computer assistance). Predominant joint systems, and their variations along the alignment, can be determined in this way. Based on surface mapping, the geologist must then project the geologic conditions to the elevation of the proposed underground structures so that tunneling conditions can be assessed.

e. Hydrogeology.

(1) Groundwater has the potential to cause great difficulties for underground work, and a special effort should be made to define the groundwater regime—aquifers, sources of water, water quality and temperature, depth to groundwater. A hydrological survey is necessary to ascertain whether tunnel construction will have a deleterious effect on the groundwater regime and the flora and fauna that depend on it. Maps and air photos, including infrared, will help define the groundwater conditions. Mapping of permanent or ephemeral streams and other water bodies and the flows and levels in these bodies at various times of the year is usually required. Proximity of the groundwater table may be judged by the types of vegetation growing on the site.

(2) As a part of the hydrogeological survey, all existing water wells in the area should be located, their history and condition assessed, and groundwater levels taken. Additional hydrogeological work to be carried out at a later stage includes measurements of groundwater levels or pressures in boreholes, permeability testing using packers in boreholes, and sometimes pumping tests.

f. Geophysical explorations from the ground surface.

(1) Geophysical methods of exploration are often useful at the earlier stages of a project because they are relatively inexpensive and can cover relatively large volumes of geologic material in a short time. Details on the planning and execution of geophysical explorations can be found in EM 1110-1-1802.

(2) The most common geophysical explorations carried out for underground works are seismic refraction or reflection and electric resistivity surveys. Seismic

explorations can measure the seismic velocity of underground materials and discover areas of velocity contrasts, such as between different kinds of rock or at fault zones. They are also useful in determining the elevation of the groundwater table.

(3) Seismic velocity is taken as a measure of rock quality and often used to assess rippability of the rock by ripper-equipped dozers. If there is no seismic velocity contrast across a boundary, the boundary will remain invisible to the seismic exploration.

(4) Depending on the energy applied in the seismic work and the particular technique, seismic explorations can be designed for shallow work with high resolution and for deep explorations with a lower resolution. Deep seismic explorations, using sophisticated computer enhancement of the signals, are regularly employed in the petroleum industry.

(5) Electrical resistivity measurements use arrays of power source and measurement points and provide an image of resistivity variations in the ground. These measurements are usually used to determine the depth to groundwater.

g. Additional explorations during feasibility studies. It is often appropriate to conduct initial field explorations in the form of borings or trenching at this early stage, primarily to verify the presence or location of critical geologic features that could affect the feasibility of the project or have a great effect on the selection of tunnel portals.

4-3. Explorations for Preconstruction Planning and Engineering

a. General.

(1) During the engineering design phases, explorations must be carried out to acquire data not only for the design of the underground structures but also for their construction. For this reason, exploration programs for underground works must be planned by engineering geologists or geotechnical engineers in close cooperation with designers and construction engineers.

(2) Most geotechnical data for design are obtained during preconstruction planning and engineering, but supplemental explorations, as well as explorations and testing for purposes of construction, may be carried out in the later design stage.

b. Environmental and geologic data requirements.

(1) The specific environmental data needs for a particular underground project very much depend on the geologic and geographic environment and the functional requirement of the underground facility. Some generalities can be stated, however, presented here in the form of a checklist:

- Existing infrastructure; obstacles underground and above.
- Surface structures within area of influence.
- Land ownership.
- Contaminated ground or groundwater.
- Naturally gassy ground or groundwater with deleterious chemistry.
- Access constraints for potential work sites and transport routes.
- Sites for muck transport and disposal.
- Legal and environmental constraints, enumerated in environmental statements or reports or elsewhere.

(2) As earlier noted, required geologic data include the geologic profile, rock and rock mass properties, hydrogeology, and exposure to geologic hazards. After initial fact finding and mapping, it is often possible to divide the tunnel alignment into zones of consistent rock mass condition. Criteria for zonation would be site specific, but factors involving intact rock, rock mass, and excavation system characteristics should be considered. Each zone should be characterized in terms of average expected condition as well as extreme conditions likely to be encountered.

(3) Initial literature work and mapping should identify major components of the stratigraphy and the geologic structure, which form the framework for zonation of the alignment and for the planning of the explorations. An appropriate rock mass classification scheme should be selected and all data necessary for the use of the classification system obtained. During construction, a more simplified system may be established that can be used by field people with little delay in the daily construction routine.

(4) Particular attention should be given to the following types of information:

- Top of rock; depth of weathered rock.
- Water bearing zones, aquifers, fault zones, and caves.
- Karstic ground conditions.
- Very strong (>250 MPa) and very abrasive material that can affect TBM performance.
- Highly stressed material with potential for overstress.
- Potential for gases.
- Corrosive groundwater.
- Slake-susceptible material and material with potential for swell.
- Material otherwise affected by water (dissolution, swell).
- Zones of weak rock (low intact strength, altered materials, faulted and sheared materials).

c. Strategies for exploration.

(1) Because of the complexities of geology and the variety of functional demands, no two tunnels are alike. It is therefore difficult to give hard and fast rules about the required intensity of explorations or the most appropriate types of exploration. Nonetheless, some common-sense rules can help in the planning of explorations.

(a) Plan explorations to define the best, worst, and average conditions for the construction of the underground works; locate and define conditions that can pose hazards or great difficulty during construction.

(b) Use qualified geologists to produce the most accurate geologic interpretation so as to form a geologic model that can be used as a framework to organize data and to extrapolate conditions to the locations of the underground structures.

(c) Determine and use the most cost-effective methods to discover the information sought (e.g., seismic refraction to determine top of rock).

(d) Anticipate methods of construction and obtain data required to select construction methods and estimate costs (e.g., data to estimate TBM performance and advance rates).

(e) Anticipate potential failre modes for the completed structures and required types of analysis, and obtain the necessary data to analyze them (e.g., in situ stress, strength, and modulus data for numerical modeling).

(f) Drill at least one boring at each shaft location and at each portal.

(g) Special problems may require additional explorations (e.g., to determine top of rock where there is a potential for mixed-face tunneling conditions or to define the extent of a pollutant plume).

(2) The complexity and size of an underground structure has a bearing on the required intensity of explorations. A long tunnel of small diameter does not warrant the expense of detailed explorations, and a tunneling method able to cope with a variety of conditions is required. On the other hand, a large underground cavern, such as an underground power house or valve chamber is more difficult to construct and warrants detailed analyses that include closely spaced borings, reliable design data, and occasionally a pilot tunnel.

(3) Frequently, even the most thorough explorations will not provide sufficient information to anticipate all relevant design and construction conditions. This happens, for example, in deposits of alluvial or estuarine origin, or in badly folded and faulted rock. Here, the variation from point to point may be impossible to discover with any reasonable exploration efforts. In such instances, the design strategy should deal with the average or most commonly occurring condition in a cost-effective manner and provide means and methods to overcome the worst anticipated condition, regardless of where it is encountered.

(4) In mountainous terrain, it is often difficult or very expensive to gain access to the ground surface above the tunnel alignment for exploratory drilling. Many tunnels have been driven with borehole data available only at the portals. In such instances, maximum use must be made of remote sensing and surface geologic mapping, with geologic extrapolations to tunnel depth. The tunnel must be designed to deal with postulated worst-case conditions that may never actually be encountered. The strategy may also include long horizontal borings drilled from the portals or probeholes drilled from the face of the advancing tunnel.

Horizontal boreholes up to 540 m (1,800 ft) long were drilled from one portal for the Cumberland Gap (Tennessee, Kentucky) highway tunnel. For the Harlan diversion tunnels in Kentucky, the USACE employed horizontal borings up to 360 m (1,080 ft) long.

(5) It may also be difficult or expensive to obtain borehole data for tunnels under rivers and beneath lakes and the ocean. A minimum of borings should still be drilled, even if costly, but maximum use should be made of subbottom profiling. For the Boston Effluent Outfall Tunnel, borings were drilled offshore about every 300-400 m (1,000-1,300 ft), and heavy use was made of seismic refraction profiling as well as deep digital reflection, at a cost of exploration approaching 10 percent of construction cost. Where large openings are required in difficult geology, pilot tunnels are often warranted.

(6) The question is frequently argued of how much information must be obtained for the design of an underground structure. The simple answer can be stated in terms of cost-effectiveness: If the next boring does not add knowledge that will reduce construction cost an amount equal to the cost of the boring, then sufficient information has already been obtained. In practice, this assessment is not so simple, because the results of the next boring, by definition, are unknown, and the construction cost saving can be assessed only on a very subjective basis.

(7) The intensity of explorations can be measured in several meaningful ways:

- Cost of full geotechnical exploration program (borings, testing, geophysics) as percentage of construction cost.
- Typical spacing of boreholes.
- Number of meters of borehole drilled for each 100 m of tunnel.

(8) The required intensity of explorations will vary at least with the following factors: complexity of geology, project environment, depth of tunnel, end use requirements of the tunnel, accessibility for explorations, and relative cost of individual borings.

(9) A practical guide for assessing the suitability of an exploration program is shown in Table 4-1. The guide starts with a relatively simple base case and employs factors up or down from there. The base case considered is a 6-m (20-ft) drainage tunnel through moderately complex geology in a suburban area at a moderate depth of about 30 m (100 ft).

d. Exploratory borings.

(1) Tools and methods for exploratory borings and sampling are described in detail in EM 1110-1-1804. The most common sample size used for core borings for underground works is the NX-size, of approximately 2-in. diam.

(2) For deep boreholes, it is common to use wireline drilling. With this method of drilling, a large-diameter drill stem is used, furnished at the bottom end with a suitable carbide or diamond bit. The core barrel is lowered to the bottom by a wireline and snaps into the drill bit while coring takes place. When a core run is finished, the core barrel is reeled up and the core withdrawn from the barrel. With this method, time-consuming trips in and out of the hole with the entire drill string are avoided. At the same time, the drill string provides borehole stability.

Table 4-1
Guidelines for Assessing Exploration Needs for Tunnels and in Rock

	Cost of Borings and Testing, % of Construction	Borehole Spacing	Borehole Length per 100 m Tunnel
Base case	0.4-0.8	150-300 m	15-25 m
Extreme range	0.3-10	15-1,000 m	5-1,000 m
For conditions noted, multiply base case by the following factors:			
Simple geology	0.5	2-2.5	0.5
Complex geology	2-3	0.3-0.5	2-3
Rural	0.5	2-2.5	0.5
Dense urban	2-4	0.3-0.4	2-5
Deep tunnel	0.8-1	Increase borehole spacing in proportion to depth of tunnel	
Poor surface access	0.5-1.5	5-10+	variable
Shafts and portals	NA	At least one each	NA
Special problems	1.5-2	0.2-0.5 locally	variable

(3) On occasions, core is extracted only from around the elevation of the underground structure; the remainder of the hole drilled blind, i.e., without core. Usually, however, the entire length of core is of geological interest and should be recovered. If a full sweep of downhole geophysical tools is run in the hole, geologic correlation between holes is usually possible, and core may be needed only at the depth of the underground structure.

4-4. Testing of Intact Rock and Rock Mass

a. General. Laboratory tests provide a quantitative assessment of the properties of intact rock specimens. Laboratory tests do not necessarily represent the properties of the rock mass in situ, which are affected by joints, bedding planes, and other flaws that are not present in the laboratory specimens. In addition, mechanisms of behavior tested in the laboratory do not always represent the mechanisms of behavior experienced in situ. Nonetheless, laboratory testing provides indices and clues to in situ behavior, as well as data for comparison and correlation with experience records. Determination of properties representative of in situ conditions and of the undisturbed rock mass may require in situ testing.

b. Tests in boreholes and trial excavations.

(1) A number of properties can only be measured by in situ tests, either in boreholes or in trial excavations or tunnels. Standardized procedures for in situ tests are published by the American Society for Testing and Materials (ASTM) and as recommendations of the International Society of Rock Mechanics, and in the Rock Testing Manual.

(2) The most common in situ tests performed for underground works are listed in Table 4-2.

Table 4-2
Common Test Methods

Parameter	Test Method
In situ stress state	U.S. Bureau of Mines Borehole Deformation Gage Hydraulic fracturing Overcoring of hollow inclusion gage
Modulus of deformation	Rigid plate loading test Flexible plate loading test Flatjack test Radial jacking test Diametrically loaded borehole jack Pressuremeter (soft rock)
Shear strength	Torsional shear test Direct shear test Pressuremeter (soft rock)
Permeability	Constant head injection test Pressure pulse technique Pumping tests

(3) Permeability tests are performed using packers to isolate intervals in boreholes; double packers insulating 10 or 20 ft (3 or 6 m) of borehole are usually used. Sometimes single packer tests are performed, isolating the lower part of the borehole. Permeability tests should be performed in every borehole wherever groundwater is a potential problem. Other tests conducted in boreholes can be performed reasonably inexpensively, while those performed in test trenches or pilot tunnels tend to be expensive.

(4) In many cases a suite of downhole geophysics surveys are also run in boreholes in rock. EM 1110-1-1802 describes the common downhole geophysical surveying techniques. A common combination of surveys performed includes the following:

- Caliper log to measure the borehole diameter and locate washouts.
- Electric resistivity to measure variations of the resistivity of the rock mass.
- Spontaneous potential to measure the potential difference between an underground location and a reference location.
- Natural gamma to measure gamma emissions from radioactive materials in the ground.

(5) Other downhole survey techniques can provide images of the borehole wall (gyroscopically controlled) and information about the density, porosity, or seismic velocity of the rock. Seismic methods using boreholes include cross-hole (hole-to-hole) methods as well as methods using a source at locations at the ground surface with geophones in the borehole, or vice versa.

c. Tests performed in the laboratory. Test procedures and standards for rock tests in the laboratory are specified in ASTM Standards, Recommendations of the International Society of Rock Mechanics, and in the Rock Testing Handbook. Some of these tests can be characterized as index tests, used mostly for correlation and comparison, while others directly measure properties important to behavior. The tests most commonly performed in the laboratory for underground works are listed in Table 4-3.

Table 4-3
Tests Performed in Laboratory

Rock Property	Parameter/Characterization
Index properties	Density Porosity Moisture content Slake durability Swelling index Point load index Hardness and abrasivity
Strength	Uniaxial compressive strength Triaxial compressive strength Tensile strength (Brazilian) Shear strength of joints
Deformability	Young's modulus Poisson's ratio
Time dependence	Creep characteristics
Permeability	Coefficient of permeability
Mineralogy and grain sizes	Thin-sections analysis Differential thermal analysis X-ray diffraction

d. Use of test data. The following indicates some particular uses of tests and test data.

(1) Rock variability.

(a) Index tests.

(b) Point load tests.

(2) Stability in homogeneous rock.

(a) Unconfined compressive strength.

(b) In situ stress.

(3) Stability in jointed rock.

(a) Rock mass index data (see later).

(b) Unconfined compressive strength.

(c) Joint shear strength.

(d) In situ stress.

(4) Groundwater flow and pressure.

(a) In situ permeability.

(b) In situ water pressure.

(c) Porosity.

(d) Pumping test data.

(5) Sensitivity to atmospheric exposure and water content change.

(a) Slake durability test.

(b) Swelling index.

(c) Density.

(d) Moisture content.

(e) Mineralogy.

(6) Computer modeling.

(a) In situ stress.

(b) Young's modulus.

(c) Poisson's ratio.

(d) Uniaxial and triaxial strength data.

(7) TBM performance (see Appendix C for details).

(a) Uniaxial compressive strength.

(b) Tensile strength.

(c) Hardness and abrasivity.

(d) Mineralogy.

e. Rock mass classification systems.

(1) Rock mass classification systems for engineering purposes use experience derived from previous projects to estimate the conditions at a proposed site. These systems combine findings from observation, experience, and engineering judgment to provide an empirically based, quantitative assessment of rock conditions. For a classification system to be successful, the parameters must be relevant to their application and be capable of being consistently rated against some set of standard descriptions or objective set of rules on the basis of simple observations or measurements.

(2) The diversity of classifications of rock material, rock mass, and rock structure used in geology and geotechnical engineering is a function not only of the variability of the rock materials and their properties but also of the use to which the classification is put. Classification systems can be used either to simply characterize some particular rock property and thereby facilitate the application of information into a design (i.e., classification of rock strength by simple index tests) or relate findings to the determination of actual design parameters (i.e., tunnel support pressure).

(3) Classification systems have proven effective for the selection of underground opening support. The complexity of geology over the length of a tunnel drive means that even the best geologic surveys of the site for a proposed tunnel are unable to provide a complete understanding of the underground conditions. The optimum approach allows the design to be modified as information from the underground becomes available. Even once the ground is known, the final loading condition will only be known approximately and will probably vary along the tunnel length and be dependent on local geology and support performance. The main rock classification systems currently used to assist in the design of underground excavations are summarized in Table 4-4. A brief description of these systems is presented in the following. The use of these classifications for selection of initial ground support is discussed in Chapter 7.

(a) *Rock load method.* The application of a classification system determining tunnel support requirements for tunnels was first proposed in the United States by Terzaghi (1946), who developed a classification system for rock loads carried by steel ribs and lagging for a variety of rock conditions. The system is based on visual descriptions of rock conditions and can still be used for tunnels where steel sets and lagging are the method of tunnel support.

(b) *RQD.* RQD (Deere et al. 1967; Deere 1968) provides a quantitative index of fracturing within a rock mass based on the recovery of drill core. RQD is an empirical index. It is determined by counting all pieces of sound core over 100 mm (4 in.) long as recovery and expressing the recovery as a percentage of the total length drilled. RQD is expressed as follows:

$$RQD\ (\%) = (\text{length of core with pieces} > 100\ \text{mm}) \times 100/\text{length of core run}$$

The index is derived from standard-sized core at least 50 mm in diameter over lengths of borehole of at least 1.5 m (5 ft) in length. Although the degree of fracturing in a rock mass is a significant factor in determining tunnel support, other geologic conditions contribute to the performance of openings. These conditions include groundwater conditions, in situ stresses, fracture condition, fracture orientation, and opening size. RQD by itself does not provide a complete method for establishing tunnel support or standup times. RQD is, however, an essential element within the framework of other rock mass classification systems. It provides a quantitative index of rock quality in terms of fracture frequency that is easily obtained and has become an accepted part of core logging procedures. Most rock mass classification systems use RQD as a parameter to define fracture intensity of a rock mass. In combination with other parameters, an overall rating is established for the rock mass that reflects support needs and stand-up times for excavations. Table 4-5 shows the basic RQD descriptions.

(c) *Rock structure rating (RSR) concept.* RSR is based on an evaluation of conditions in 53 tunnel projects. It is a quantitative method for describing the quality of a rock mass and for selecting appropriate ground support, primarily steel ribs. Factors related to geologic conditions and to construction are grouped into three basic parameters, A, B, and C (Wickham, Tiedemann, and Skinner 1972; Skinner 1988). Parameter A is a general appraisal of the rock structure through which the tunnel is driven, determined on the basis of rock type origin, rock hardness, and geologic structure. Parameter B describes the effect of discontinuity pattern with respect to the direction of tunnel drive on the basis of joint spacing, joint orientation, and direction of tunnel drive. Parameter C includes the effect

Table 4-4
Major Rock Classification Systems Currently in Use (Barton 1988)

Name of Classification	Originator and Date	Country of Origin	Application
Rock Loads	Terzaghi (1946)	United States	Tunnels with steel supports
Stand-up Time	Lauffer (1958)	Austria	Tunneling
RQD	Deere et al. (1967) Deere (1968)	United States	Core logging, tunneling
RSR Concept	Wickham et al. (1972)	United States	Tunnels with steel supports
Geomechanics (RMR)	Bienawski (1979)	S. Africa	Tunnels, mines
Q-System	Barton et al. (1974)	Norway	Tunnels, large chambers

Table 4-5
Descriptions of Rock Quality Based on RQD (From Deere and Deere 1988)

RQD, percent	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

of groundwater inflow on the basis of overall rock mass quality, joint condition, and groundwater inflow. The RSR value of any tunnel section is obtained by summing the numerical values determined for each parameter. RSR is as follows:

$$RSR = A + B + C$$

The values for Parameters A, B, and C are given in Chapter 7 together with the estimate of support requirements in terms of an index, Rib Ratio (RR).

(d) *Geomechanics rock mass classification system.* The Geomechanics Rock Mass Classification System provides a quantitative method for describing the quality of a rock mass, selecting the appropriate ground support, and estimating the stand-up time of unsupported excavations. It is based on the summation of ratings for the following six rock mass parameters: strength of intact rock material, RQD, spacing of joints, condition and quality of joints and discontinuities, condition of groundwater, and orientation of joint or discontinuity relative to the excavation. The ratings for the parameters are provided in Appendix C.

(e) *Rock mass quality.* This system covers the whole spectrum of rock mass qualities from heavy squeezing ground to sound unjointed rocks. The system uses six parameters to describe the rock mass quality (Q) combined as follows:

$$Q = RQD / J_n \cdot J_r / J_a \cdot J_w / SRF$$

where

RQD = rock quality designation

J_n = joint set number

J_r = joint roughness number (of least favorable discontinuity set)

J_a = joint alteration number (of least favorable discontinuity set)

J_w = joint water reduction factor

SRF = stress reduction ratio

The three ratios that comprise the rock mass quality, Q, are crude measures of physical conditions defining the rock mass. RQD/J_n is a geometry index that can be considered as a measure of block size. J_r/J_a is a shear strength index that measures interblock strength. J_w/SRF is an external stress index and is a measure of the active stress. The range of values for the parameters are provided in Chapter 7.

f. Exploration and testing for gases in the ground.

(1) Gas sampling and testing during geotechnical explorations are required if gassy ground, either naturally occurring or contaminated, is suspected in the project area. Gaseous conditions must be identified in advance so they can be accounted for in the design and mitigated during construction. Several methods for gas testing are available. Some of the gases such as hydrogen sulfide or methane can be extremely toxic and/or explosive. It is important that professionals with experience in the methods and familiar with safety regulations, hazardous levels of flammable, explosive, and toxic gasses, and emergency response procedures for both workers and the public perform the testing and sampling.

(2) Exploratory drilling where there is a potential presence of methane, hydrogen sulfide, or other gases is commonly done by practitioners in the oil and gas industry and environmental geotechnical engineering.

g. Large-scale explorations.

(1) Many types of explorations can be classified as large-scale explorations. Some of these can be useful for underground works, but most are carried out for other purposes, as described in the following.

(a) Test pits and trenches are often excavated for foundation explorations, including dam foundations. They can be useful at tunnel portal locations, where drilling can be difficult and seismic surveys ambiguous.

(b) Test blasting is useful for quarry development.

(c) Test pumping is often carried out for deep excavations to determine overall permeability and probable yield of pumping for dewatering. It is often useful for shaft explorations and sometimes for tunnels in soft ground.

(d) Test grouting is useful for planning dam foundation grouting and has occasionally been useful when the designer has determined that grouting will be an essential part of a tunnel project (e.g., to avoid ground loss and deleterious settlement).

(e) Large-diameter boreholes (e.g., calyx holes) permit inspection of the borehole walls. Such boreholes have been successful for dam and power plant explorations in the past and may still be useful, though rarely carried out.

(f) Adits and pilot tunnels are frequently used for explorations of rock quality in dam abutments and foundations and for large tunnels and chambers. Such large-diameter explorations are necessary to conduct in situ tests such as flatjack, plate jacking, or radial jacking tests and helpful for other in situ tests. In addition to providing detailed geologic information, pilot tunnels permit evaluations to be made of the excavation effort, ground support needs, sensitivity of the rock to weathering, and other construction features. If excavated in the crown of a large excavation, a pilot tunnel can be used to drain formation water, provide a path for ventilation, permit prereinforcement of poor ground, and otherwise be helpful for the completion of the work.

(2) Extrapolations of ground behavior (especially conditions such as potentially squeezing ground), from the small scale of the pilot tunnel to the full prototype, must be accomplished with care due to the difficulty in selecting scale factors. Pilot tunnels should be considered, if not always carried out, for all large underground openings. Pilot tunnels have been carried out for the Peachtree subway station in Atlanta; highway tunnels in Glenwood Canyon, Colorado, and Cumberland Gap, Tennessee; and for highway H-3 tunnels on Oahu.

4-5. Presentation of Geotechnical Data

a. It is essential to make all geotechnical information available to the contractors who are bidding for the project. EM 1110-1-1804 sets forth principles and procedures for presenting geologic and geotechnical data in contract documents. Because of the volume and complexity of the complete exploration and testing documentation, it is not usually feasible or proper to incorporate all data in the contract documents. A selection of data to be presented in the contract documents must be made for each project, depending on the importance of the data. The remainder of the data would be available for review. At a minimum, all boring logs, test trenches, and adit data should be included in the contract documents.

b. A geotechnical design summary report (GDSR) may be included in the contract documents. This report presents the design team's best estimate concerning ground conditions to be encountered and how the geotechnical data has affected the design. This report becomes the baseline against which contractor claims for differing site conditions are gaged; it must therefore be written carefully and reviewed by people knowledgeable about the contractual use of this document. Further description of the use of the GDSR is found in ASCE (1991).

4-6. Geologic Investigations During Construction

a. Additional geotechnical information is sometimes required during the construction of the underground facility for one or more of the following purposes:

- Exploration ahead of the advancing face to discover regions of potential high water inflow, very poor ground, limestone caves, buried valleys, or dips in the weathering profile.

- Classification of rock mass to determine or verify initial ground support selection.

- Verification of conditions assumed for final tunnel lining design, including choice of unlined tunnel.

- Mapping for the record, to aid in future operations, inspections, and maintenance work.

b. Exploration ahead of the face is usually performed using a percussion drill to a distance greater than the typical daily advance. The advance rate of the drill is recorded. The drill is stopped from time to time to check the water flow into the borehole. If there is a possibility of encountering water under high pressure, drilling may have to be done through a packer, or the driller must be shielded against a high-pressure water jet.

c. Probehole drilling can often be accomplished during the period of blasthole drilling. When using a TBM, the machine usually must be stopped while drilling probeholes. Unless probehole drilling can be fitted into the maintenance schedule when the machine is stopped for other purposes, probehole drilling can reduce TBM

operating time. If probehole drill steel gets stuck within the tunnel profile and cannot be recovered, then TBM advance can be severely hampered. It is, therefore, often the practice to drill over the crown of the TBM at a 3- to 6-deg angle from the tunnel axis.

d. If initial ground support is selected on the basis of ground conditions actually encountered, then a geologic appraisal is required after each round of blasting or more or less continuously for a TBM tunnel. A complete mapping in accordance with the Q method is tedious, time-consuming, and usually unnecessary. A simpler classification system, based on the characteristics of the geologic materials at hand, will usually suffice.

e. If mapping is required, it should be performed while the rock is still fresh and uncovered by debris, dust, or construction material. At the same time, the geologist should never venture into the heading of the tunnel before the heading is made safe. When initial ground support includes shotcrete placed by robot or consists of precast segmental concrete lining, mapping is not feasible. Methods of mapping are described in EM 1110-1-1804.

Chapter 5
Construction of Tunnels and Shafts

5-1. General

a. The design team must be composed of design and construction engineers and geologists experienced in underground construction. Methods and sequences of excavation affect the loads and displacements that must be resisted by initial and permanent ground support. The basic shape of an excavated opening must be selected for practicality of construction. Although it is good practice to leave many details of construction for the contractor to decide, it is often necessary for the designer to specify methods of construction when the choice of methods affects the quality or safety of the work or when construction will have environmental effects. There are aspects of construction where the design team may have to work closely with the contractor or include restrictive provisions in the specifications.

b. The basic components of underground construction include the following:

- Excavation, by blasting or by mechanical means.
- Initial ground support.
- Final ground support.

c. In the past, the terms "primary" and "secondary" support have been used for "initial" and "final" support. This usage is discouraged because it is misleading since in terms of end function, the final support has the primary role, and initial ground support is often considered temporary. However, in many instances today, initial ground support may also serve a function in the permanent support.

d. Other important components of construction include the following:

- Site and portal preparation.
- Surveying.
- Ventilation of the underground works.
- Drainage and water control.
- Hazard prevention.
- Controlling environmental effects.

These topics are discussed in this chapter; however, it is not the intent to present a complete guide to tunnel construction. The designer may have reason to explore in greater depth certain details of construction, such as blasting effects or TBM feasibility or projected advance rates.

5-2. Tunnel Excavation by Drilling and Blasting

While TBMs are used in many tunneling projects, most underground excavation in rock is still performed using blasting techniques. The design team should specify or approve the proposed method of excavation.

a. The excavation cycle. The typical cycle of excavation by blasting is performed in the following steps:

(1) Drilling blast holes and loading them with explosives.

(2) Detonating the blast, followed by ventilation to remove blast fumes.

(3) Removal of the blasted rock (mucking).

(4) Scaling crown and walls to remove loosened pieces of rock.

(5) Installing initial ground support.

(6) Advancing rail, ventilation, and utilities.

b. Full- and partial-face advance.

(1) Most tunnels are advanced using full-face excavation. The entire tunnel face is drilled and blasted in one round. Blastholes are usually drilled to a depth somewhat shorter than the dimension of the opening, and the blast "pulls" a round a little shorter (about 90 percent with good blasting practice) than the length of the blastholes. The depth pulled by typical rounds are 2 to 4 m (7-13 ft) in depth. Partial-face blasting is sometimes more practical or may be required by ground conditions or equipment limitations. The most common method of partial-face blasting is the heading-and-bench method, where the top part of the tunnel is blasted first, at full width, followed by blasting of the remaining bench. The bench can be excavated using horizontal holes or using vertical holes similar to quarry blasting. There are many other variations of partial-face blasting, such as a center crown drift, followed by two crown side drifts, then by the bench in one, two, or three stages.

(2) Reasons for choosing partial as opposed to full-face blasting include the following:

(a) The cross section is too large for one drill jumbo for example: Underground openings of the sizes usually required for powerhouses, valve chambers, and two- or three-lane highway tunnels are usually excavated using partial-face blasting excavation.

(b) The size of blast in terms of weight of explosives must be limited for vibration control.

(c) The ground is so poor that the full width of excavation may not be stable long enough to permit installation of initial ground support.

c. Design of a blasting round.

(1) The individual blasting rounds are usually designed by a blasting specialist in the contractor's employ. The design is reviewed by the engineer for compliance with specifications. Information about the detailed design of blasting rounds can be found, for example, in Langefors and Kihlstrom (1978) or Persson, Holmberg, and Lee (1993). Information about blasting agents and blasting design can also be found in handbooks published by explosives manufacturers, such as Blaster's Handbook (Dupont). See also EM 1110-2-3800. Blastholes are usually drilled using hydraulic percussion drills. The efficiency and speed of hole drilling has been improving rapidly, and bit wear and precision of drilling have also improved due to new designs of drill rods and bits. Drilling for small tunnels is often done with a single drill, but more often drill jumbos are used with two or more drills mounted. The jumbos can be rail, tire, or track mounted. Track-mounted straddle jumbos permit mucking equipment to move through the jumbo to and from the face.

(2) Effective blasting design requires attention to the degree of confinement for the detonation of each blast hole. If a blast hole is fully confined, the detonation may result merely in plastic deformation. With a nearby free face, the blast wave will create fractures toward the face, fragment the rock between the hole and the face, and remove the fragments. The distance to the free face, the burden, is taken generally between 0.75 and 1.0 times the hole spacing.

(3) In a tunnel, there is initially no free face parallel to the blasthole. One must be created by the blast design and this is done in one of several ways.

(a) The V-cut or fan-cut uses a number of holes drilled at an angle toward each other, usually in the lower middle of the face, to form a wedge. Detonation of these holes first will remove the material in the wedge and allow subsequent detonations to break to a free face.

(b) The burn cut uses parallel holes, most often four holes close together with only two loaded, or one or two large-diameter holes, usually up to 125 mm (5 in.) in diameter, unloaded. Remaining holes are laid out and initiated so that each new detonation in one or more blastholes always will break to a free face. The holes set off just after the cut are the stopping holes, also called easer, relief (reliever), or enlarger holes. The last holes to be detonated are the contour or trim holes around the periphery. The ones in the invert are called lifters.

(c) Perimeter holes are usually drilled with a lookout, diverging from the theoretical wall line by up to about 100 mm (4 in.) since it is not possible to drill right at the edge of the excavated opening. The size of the drill equipment requires a setback at an angle to cover the volume to be excavated. Successive blasts result in a tunnel wall surface shaped in a zigzag. Therefore, overbreak is generally unavoidable.

(d) Delays, electric or nonelectric, are used to control the sequence and timing of the detonations and to limit the amount of explosives detonated at any time. These are of several types. Millisecond delays are fast, ranging from 25 to 500 ms; other delays are slower. Up to 24 ms delays are available. Delays must be selected such that the rock fragments are out of the way before the next detonation occurs. Millisecond delays are often used within the burn part of the blast, with half-second delays used for the remainder. In the past, the blast was usually initiated electrically, using electrical blasting caps or initiators. Nonelectrical blast initiators and delays are now available and are often preferred because they are not affected by stray electric currents.

(e) Blasting agents are available for special purposes. They vary in charge density per length of hole, diameter, velocity of detonation, fume characteristics, water resistance, and other characteristics. In dry rock, the inexpensive ANFO (a mixture of ammonium nitrate and fuel oil) is often used. Trim holes require special blasting agents with a very low charge per meter. Blastholes are typically 45 to 51 mm (1.9-2 in.) in diameter. Sticks or sausages of explosive agents are usually 40 mm (1.6 in.) in diameter and are tamped in place to fill the hole, while those used

for trim holes are often 25 mm (1 in.) in diameter and are used with stemming.

(f) Two parameters are often calculated from a blast design: the powder factor or specific charge (kilograms of explosives per cubic meter of blasted rock) and the drill factor (total length of drill holes per volume of blasted rock (meter/cubic meter)). These are indicators of the overall economy of blasting and permit easy comparison among blast patterns. The powder factor varies greatly with the conditions. It is greater when the confinement is greater, the tunnel smaller, or when the rock is harder and more resilient. Rocks with voids sometimes require large powder factors. For most typical tunnel blasting, the powder factor varies between 0.6 and 5 kg/m^3. The powder factor can vary from 1 kg/m^3 in a tunnel with an opening size greater than 30 m^2 to more than 3 kg/m^3 for a size less than 10 m^2, in the same type of ground. Typical drill factors vary between 0.8 and 6 m/m^3. Figure 5-1 shows a typical, well-designed round. This 19.5-m^2 round uses 40 holes with a powder factor of 1.9 kg/m^3 and a drill factor of 2.2 m/m^3. Typical powder factors and drill hole requirements are shown on Figures 5-2 and 5-3.

d. Controlled blasting.

(1) The ideal blast results in a minimum of damage to the rock that remains and a minimum of overbreak. This is achieved by controlled blasting. Control of rock damage and overbreak is advantageous for many reasons:

(a) Less rock damage means greater stability and less ground support required.

(b) The tunneling operations will also be safer since less scaling is required.

(c) Less overbreak makes a smoother hydraulic surface for an unlined tunnel.

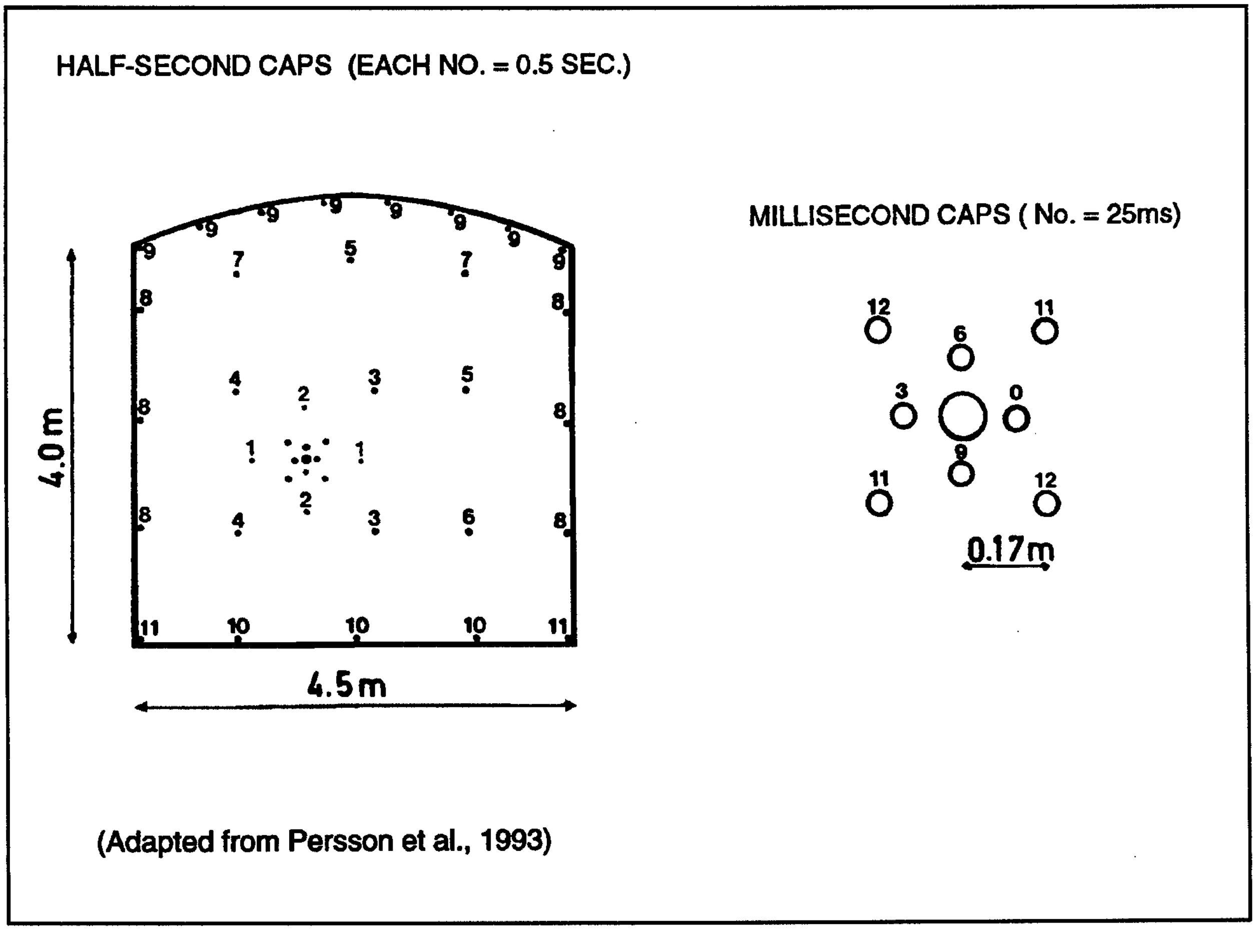


Figure 5-1. Blasting round with burn cut blastholes 3.2 m, advance 3.0 m

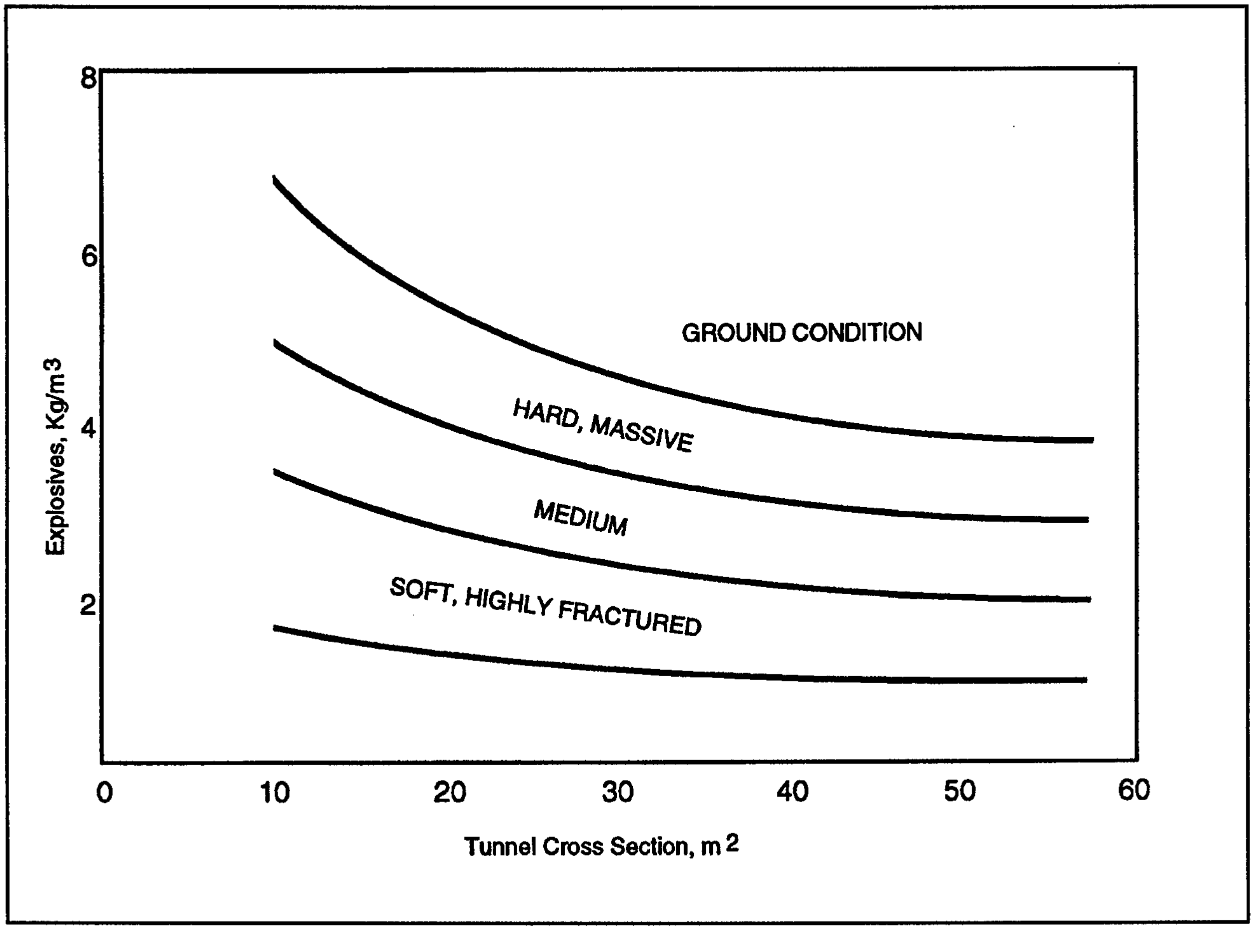


Figure 5-2. Typical powder factors

(d) For a lined tunnel, less overbreak means less concrete to fill the excess voids.

(2) Controlled blasting involves a closer spacing of the contour or trim holes, which are loaded lighter than the remainder of the holes. A rule often used is to space contour holes about 12-15 times hole diameter in competent rock, and 6-8 times hole diameter in poor, fractured rock. Because controlled blasting generally requires more blast holes than otherwise might be required, it takes longer to execute and uses more drill steel. For these reasons, contractors are often reluctant to employ the principles of controlled blasting.

(3) But controlled blasting requires more than just the design of proper perimeter blasting. Blast damage can occur long before the trim holes are detonated. Controlled blasting requires careful design and selection of all aspects of the round-geometry, hole diameter, hole charges, hole spacings and burdens, and delays—as well as careful execution of the work.

(4) One of the keys to successful controlled blasting is precise drilling of blast holes. Deviations of blastholes from their design locations quickly lead to altered spacings and burdens, causing blast damage and irregular surfaces. Modern hydraulic drills are not only quick but also permit better precision than was the norm. The highest precision is obtained with the use of computer-controlled drill jumbos in homogeneous rock.

(5) Inspection of the blasted surfaces after the blast can give good clues to the accuracy of drilling and the effectiveness with which blasting control is achieved. A measure of success is the half-cast factor. This is the ratio of half casts of blast holes visible on the blasted surface to

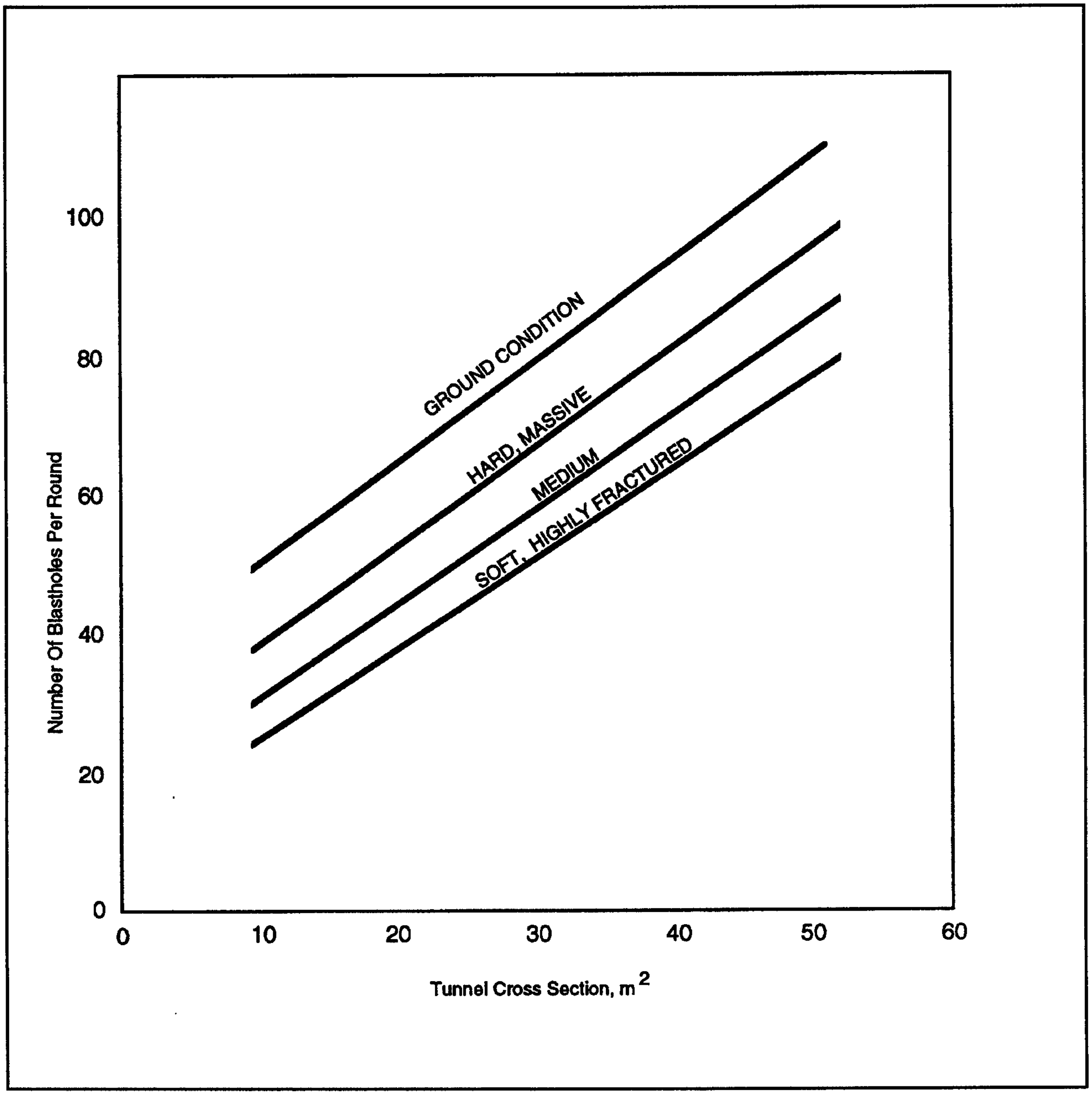


Figure 5-3. Typical drill hole requirements

the total length of trim holes. Depending on the quality of the rock and the inclination of bedding or jointing, a half-cast factor of 50 to 80 percent can usually be achieved. Irregularities in the surface caused by imprecise drilling are also readily visible and measurable. The regularity and appropriateness of the lookout should also be verified. Other means to verify the quality of blasting include methods to assess the depth of blast damage behind the wall. This may be done using seismic refraction techniques and borescope or permeability measurements in cored boreholes. The depth of the disturbed zone can vary from as little as 0.1-0.2 m (4-8 in.) with excellent controlled blasting to more than 2 m (7 ft) with uncontrolled blasting.

e. Blast vibrations. Blasting sets off vibrations that propagate through the ground as displacement or stress waves. If sufficiently intense, these waves can cause damage or be objectionable to the public. Vibration control is

particularly important in urban environments. Monitoring and control of blasting are described in detail in several publications, including Dowding (1985).

(1) The intensity of blasting vibrations felt a given distance from the blast is a function of the following factors:

- The total charge set off by each individual delay (a delay as small as 8 ms is sufficient to separate two detonations so that their blast wave effects do not overlap).
- The distance from the detonation to the point of interest.
- The character of the ground (high-modulus rock permits the passage of waves of higher frequency, which quickly damp out in soil-like materials).
- The degree of confinement of the blast (the greater the confinement, the greater percentage of the total energy will enter the ground as vibration energy).
- Geometric site features will sometimes focus the vibration energy, as will geologic features such as bedding with hard and soft layers.

With a given explosive charge and a given distance, the intensity of vibration can be estimated using scaling laws. Most commonly, the square-root scaling law is used, which says that the intensity of the vibration is a function of the square root of the charge, W. The most important vibration parameter is the peak particle velocity, V.

$$V = H\,(D/W^{1/2})^{-B}$$

where B is an empirically determined power. The quantity $D/W^{1/2}$ is called the scaled distance, and H is the peak velocity at a scaled distance of one. This relationship plots as a straight line on a log-log plot of velocity against scaled distance, with D in meters, W in kilograms of explosive, V in millimeters/second. The quantity H varies with blast characteristics, confinement, and geologic environment. A typical range for H is 100 to 800 (metric); for a given geologic medium, H can vary within a single blast: 250 for the V-cut, 200 for production holes, and 150 for the trim holes. H is generally smaller for shorter rounds. The power B can vary from 0.75 to 1.75; it is often taken as 1.60. For a particular site or environment, the empirical relationship can be developed based on trial blasts, using the log-log plot. Many factors affect the measured vibrations and a precise relationship is not likely to evolve. Rather, ranges of data are used to develop a safe envelope for production blasting. Typical ranges of peak particle velocity as a function of scaled distance are shown on Figure 5-4. A typical relationship between allowable charge per delay and distance for a vibration limit of 50 mm/s (2 in./s) is shown in Table 5-1 (SME 1992).

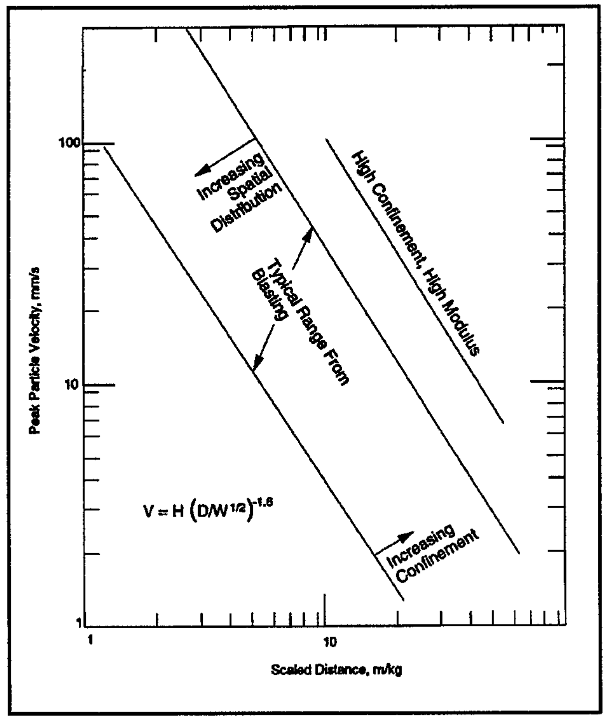


Figure 5-4. Ground vibrations from blasting

Table 5-1
Allowable Change per Delay

Allowable Charge, lb	Distance, ft
0.25	10
1.0	20
6	50
25	100
156	250

(2) Damage to structures caused by blasting is related to peak particle velocity. It is generally recognized that a peak particle velocity of 50 mm/s (2 in./s) will not damage residential structures or other buildings and facilities. In fact, most well-built structures can withstand particle

velocities far greater than 50 mm/s (2 in./s); however, it is the generally accepted limit for blasting vibrations.

(3) When blasting is carried out in the vicinity of fresh concrete, peak velocities must be restricted to avoid damage to the concrete. This concern is discussed in some detail in the Underground Mining Methods Handbook (SME 1992). Both structural concrete and mass concrete are relatively insensitive to damage when cured. Concrete over 10 days old can withstand particle velocities up to 250 mm/s (10 in./s) or more. Very fresh concrete that has not set can withstand 50 mm/s (2 in./s) or more. On the other hand, young concrete that has set is subject to damage. The peak particle velocity in this case may have to be controlled to under 6 mm/s (0.25 in./s), and particle velocities should not exceed 50 mm/s (2 in./s) until the concrete is at least 3 days old. These values may vary with the character of the foundation rock, the setting time and strength of the concrete, the geometry of the structure, and other characteristics. For important structures, site-specific analysis should be conducted to set blasting limits.

(4) Damage of intact rock in the form of microfractures usually does not occur below particle velocities of 500-1,000 mm/s, depending on the strength of the rock.

(5) Human perception is far more sensitive to blasting vibrations than are structures. Vibrations are clearly noticeable at peak particle velocities as low as 5 mm/s (0.2 in./s) and disturbing at a velocity of 20 mm/s (0.8 in./s). Perception of vibrations is, to a degree, a function of the frequency of the vibrations; low-frequency vibrations (<10-15 Hz) are more readily felt than high-frequency vibrations. Furthermore, vibrations may be much more objectionable during night hours. Setting acceptable blasting limits in an urban area requires adherence to locally established codes and practice. If codes do not exist, public participation may be required in setting peak velocity limits. The U.S. Bureau of Mines (Siskind et al. 1980) has made recommendations on peak particle velocities as shown on Figure 5-5 that may be used when no local ordinances apply. Two examples of blasting limits in urban areas follow:

(a) For construction of the TARP system in Chicago, blasting was limited to the hours of 8 a.m. and 6 p.m. Peak particle velocity at inhabited locations were limited to 12.5 mm/s (0.5 in./s) for the frequency range of 2.6-40 Hz; 18.75 mm/s (0.75 in./s) for the range above 40 Hz, and lower than 12.5 mm/s (0.5 in./s) for frequencies under 2.6 Hz. These kinds of restrictions resulted in contractors generally choosing mechanical excavation methods rather than blasting for shafts.

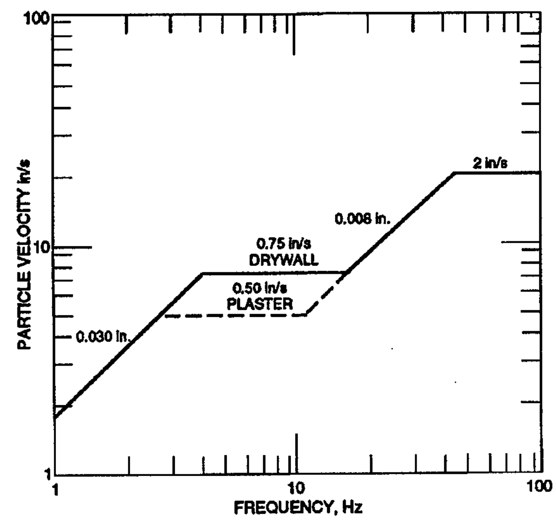


Figure 5-5. U.S. Bureau of Mines recommended blasting level criteria

(b) For MARTA construction in Atlanta, peak velocity was restricted to 25 mm/s (1 in./s) at the nearest inhabited structure and 50 mm/s (2 in./s) at the nearest uninhabited structure. Between 10 p.m. and 7 a.m., velocities were limited to 15 mm/s (0.6 in./s). Air blast overpressures were also restricted.

f. Mucking.

(1) Muck removal requires loading and conveying equipment, which can be trackless (rubber tired, in shorter tunnels) or tracked (rail cars, in longer tunnels) or belt conveyors. Provisions for passing trains or vehicles must be provided in long tunnels. Because of the cyclic nature of blasting excavation, great efficiency can be achieved if crews and equipment can work two or more tunnel faces at the same time.

(2) The ideal blast results in breaking the rock such that few pieces are too large to handle; however, excessive fines usually mean waste of explosive energy. The muck pile can be controlled by the timing of the lifter hole detonation. If they are set off before the crown trim holes, the pile will be compact and close to the face; if they are set off last, the pile will be spread out, permitting equipment to move in over the muck pile.

g. Scaling. An important element of excavation by blasting is the scaling process. Blasting usually leaves behind slivers or chunks of rock, loosened and isolated by

blast fractures but remaining tenuously in place. Such chunks can fall after a period of some time, posing significant danger to personnel. Loose rock left in place can also result in nonuniform loads on the permanent lining. Loosened rock is usually removed by miners using a heavy scaling bar. This work can be dangerous and must be conducted with great care by experienced miners. Tools are now available to make this a much less dangerous endeavor. Hydraulically operated rams or rock breakers can be mounted at the end of a remotely operated hydraulic arm. This greatly reduces the hazard and may improve the speed with which this task is accomplished.

5-3. Tunnel Excavation by Mechanical Means

Much underground excavation today is performed by mechanical means. Tools for excavation range from excavators equipped with ripper teeth, hydraulic rams, and roadheaders to TBMs of various designs. By far, TBMs are the most popular method of excavation. Roadheaders are versatile machines, useful in many instances where a TBM is not cost-effective. This section describes roadheader and TBM excavation methods and the factors that affect the selection of mechanical excavation methods.

a. Roadheader excavation. Roadheaders come in many sizes and shapes, equipped for a variety of different purposes. They are used to excavate tunnels by the full-face or the partial-face method, and for excavation of small and large underground chambers. They may also be used for TBM starter tunnels, ancillary adits, shafts, and other underground openings of virtually any shape and size, depending on rock hardness limitations. Most roadheaders include the following components:

- Rotary cutterhead equipped with picks.
- Hydraulically operated boom that can place the cutterhead at a range of vertical locations.
- Turret permitting a range of horizontal motion of the cutterhead.
- Loading device, usually an apron equipped with gathering arms.
- Chain or belt conveyor to carry muck from the loading device to the rear of the machine for off-loading onto a muck car or other device.
- Base frame, sometimes with outriggers or jacks for stabilization, furnished with electric and hydraulic controls of the devices and an operator's cab.
- Propelling device, usually a crawler track assembly.

A typical, large roadheader is shown on Figure 5-6.

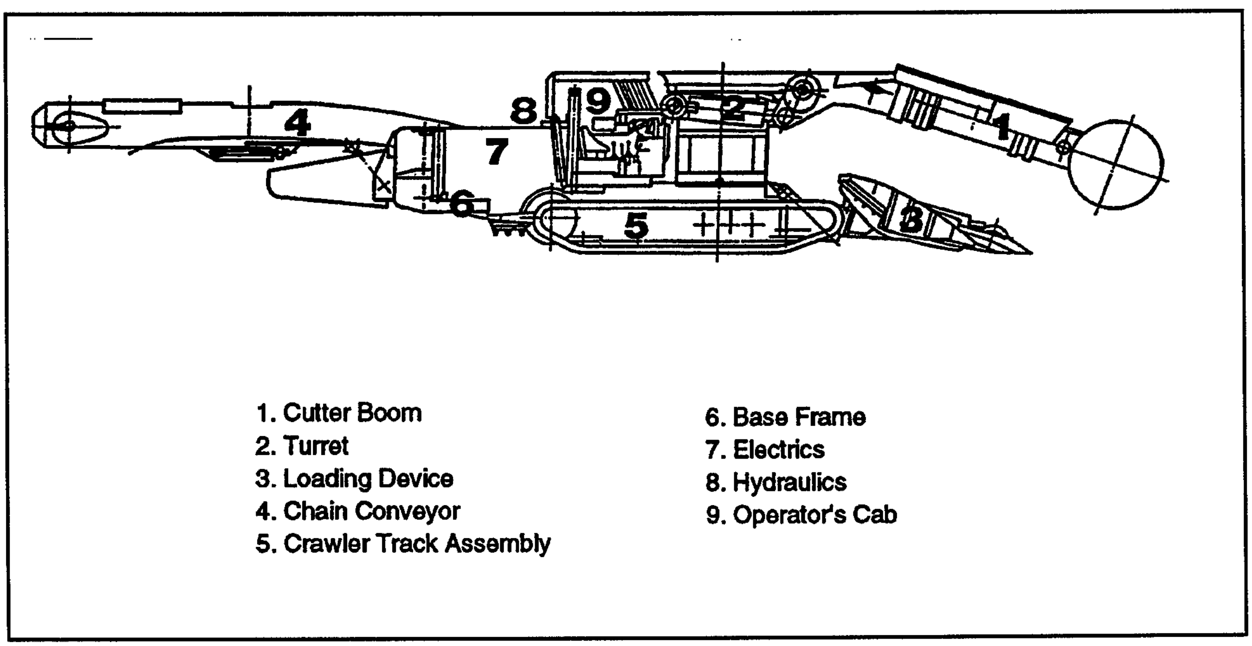


Figure 5-6. Alpine Miner 100

(1) Several types and sizes of cutterheads exist. Some rotate in an axial direction, much like a dentists drill, and cut the rock by milling as the boom forces the cutterhead, first into the face of the tunnel, then slewing horizontally or in an arch across the face. Others rotate on an axis perpendicular to the boom. The cutterhead is symmetrical about the boom axis and cuts the rock as the boom moves up and down or sideways. The cutterhead is equipped with carbide-tipped picks. Large radial drag picks or forward attack picks are used, but the most common are the point attack picks that rotate in their housings. The spacing and arrangement of the picks on the cutterhead can be varied to suit the rock conditions and may be equipped with high-pressure water jets in front of or behind each pick, to cool the pick, improve cutting, remove cuttings, and suppress dust generation. Depending on the length of the boom and the limits of the slewing and elevating gear, the cutterhead can reach a face area of roughly rectangular or oval shape. The largest roadheaders can cut a face larger than 60 m^2 from one position. Booms can be extended to reach further, or can be articulated to excavate below the floor level, or may be mounted on different bodies for special purposes, such as for shaft excavation, where space is limited.

(2) Most roadheaders can cut rock with an unconfined compressive strength of 60 to 100 MPa (10,000-15,000 psi). The most powerful can cut rock with a strength of 150 MPa (22,000 psi) to 200 MPa (30,000 psi) for a limited duration. Generally, roadheaders cut most effectively into rocks of a strength less than 30 MPa (5,000 psi), unless the rock mass is fractured and bedded. The cutting ability depends to a large measure on the pick force, which again depends on the torque available to turn the cutterhead, the cutterhead thrust, slewing, and elevating forces. The advance rate depends on the penetration per cut and the rotary speed of the cutterhead. The torque and speed of the cutterhead determines the power of the head. Cutting hard rock can be dynamic and cause vibrations and bouncing of the equipment, contributing to component wear; therefore, a heavy, sturdy machine is required for cutting hard rock. Typical small-to-medium roadheaders weigh about 20 to 80 tons and have available cutterhead power of 30 to 100 KW, total power about 80 to 650 KW. The larger machines weigh in excess of 90 tons, with cutterhead power of up to 225 KW. With a well-stabilized roadheader body, a cutterhead thrust of more than 50 tons can be obtained.

(3) Roadheader performance in terms of excavation rate and pick consumption can be predicted based on laboratory tests. Types of tests and examinations typically performed include thin-section analysis to determine the cementation coefficient and the quartz content and Shore scleroscope and Schmidt hammer hardness tests. Density, porosity, compressive, and tensile strength tests are also useful. Bedding and jointing also affect the efficiency of cutting. In a heavily jointed mass, ripping and loosening of the jointed mass can be more important than cutting of the intact rock. Bedding planes often facilitate the breaking of the rock, depending on the direction of cutterhead rotation relative to the bedding geometry. An experienced operator can take advantage of the observed bedding and jointing patterns to reduce the energy required to loosen and break the rock, by properly selecting the pattern and sequence of excavating the face. The selection of equipment should be made without regard to the potential benefits from the bedding and jointing. The equipment should be capable of cutting the intact rock, regardless of bedding and jointing.

b. Excavation by tunnel boring machine. A TBM is a complex set of equipment assembled to excavate a tunnel. The TBM includes the cutterhead, with cutting tools and muck buckets; systems to supply power, cutterhead rotation, and thrust; a bracing system for the TBM during mining; equipment for ground support installation; shielding to protect workers; and a steering system. Back-up equipment systems provide muck transport, personnel and material conveyance, ventilation, and utilities.

(1) The advantages of using a TBM include the following:

- Higher advance rates.
- Continuous operations.
- Less rock damage.
- Less support requirements.
- Uniform muck characteristics.
- Greater worker safety.
- Potential for remote, automated operation.

(2) Disadvantages of a TMB are the fixed circular geometry, limited flexibility in response to extremes of geologic conditions, longer mobilization time, and higher capital costs.

(3) A database covering 630 TBM projects from 1963 to 1994 has been assembled at The University of Texas at Austin (UT). This database supplies information on the

range of conditions and performance achievements by TBMs and includes 231 projects from North America, 347 projects from Europe, and 52 projects from other locations. A brief summary of the database is presented in Table 5-2. In addition, this database includes information on site geology and major impacts on construction. These are summarized in Table 5-3. Most database projects were excavated in sedimentary rock, with compressive strength between 20 and 200 MPa.

Table 5-2
Description of Projects in the UT Database

Description		Number Among Database Projects
No. of Projects in Completion Date Interval (total 630 projects)	1963-1970	26
	1971-1975	53
	1976-1980	122
	1981-1985	139
	1986-1990	176
	1991-1994	114
Total Project Lengths, km (total tunnel length in data base = 2,390 km)	1963-1970	81
	1971-1975	134
	1976-1980	400
	1981-1985	530
	1986-1990	666
	1991-1994	579
No. Projects in Excavated Diameter Interval, m	2 to 3.5 m	219
	3.6 to 5.0 m	237
	5.0 to 6.5 m	104
	6.5 to 8.0 m	36
	>8.0 m	34
No. Projects in Shaft Depth Interval	No shafts	402
	<15 m	35
	15 to 50 m	92
	>50 m	101
No. Projects in Gradient Interval	>+20% uphill	40
	+10 to +20%	6
	+3 to +10%	1
	+ 3 to - 3 %	573
	-3 to -10%	3
	-10 to -20%	7
	>-20% down	0
No. TBMs in Indicated Starting Condition	New	318
	Direct Reuse	22
	Refurbished	261
	Unspecified	29
No. TBMs with Indicated Shield Types	Open	512
	Single Shield	56
	Double Shield	38
	Special Shield	15
	Unspecified	9

Table 5-3
Description of Rock and Problems Encountered on Projects in the UT Database

Descriptor		Among Database Projects
Predominant Geology (% of projects)	Sedimentary	60%
	Metamorphic	30%
	Igneous	10%
Uniaxial Compressive Strength, MPa [96 average] [3 - 300 range]	<20 MPa	11%
	20-80 MPa	28%
	80-200 MPa	52%
	>200 MPa	9%
Projects with Special Problems		Number of Projects
Mucking capacity limitation		7
Excessive cutter wear		18
Gassy ground		25
Wide range in rock strength		43
Wide range in rock mass quality		108
Wide range in both rock strength and rock mass quality		14
High water inflow		23
Soil/weathered material		14
Major fracture zones		33
Overstressed rock		7
Major equipment breakdown		30
Contract stoppage		9

c. TBM system design and operation. A TBM is a system that provides thrust, torque, rotational stability, muck transport, steering, ventilation, and ground support. In most cases, these functions can be accomplished continuously during each mining cycle. Figure 5-7 is a sketch of a typical open or unshielded TBM designed for operation in hard rock. The TBM cutterhead is rotated and thrust into the rock surface, causing the cutting disc tools to penetrate and break the rock at the tunnel face. Reaction to applied thrust and torque forces may be developed by anchoring with braces (grippers) extended to the tunnel wall, friction between the cutterhead/shield and the tunnel walls, or bracing against support installed behind the TBM.

d. TBM performance parameters.

(1) TBM system performance is evaluated using several parameters that must be defined clearly and used consistently for comparative applications.

(a) *Shift time.* Some contractors will use 24-hr shifting and maintain equipment as needed "on the fly." As

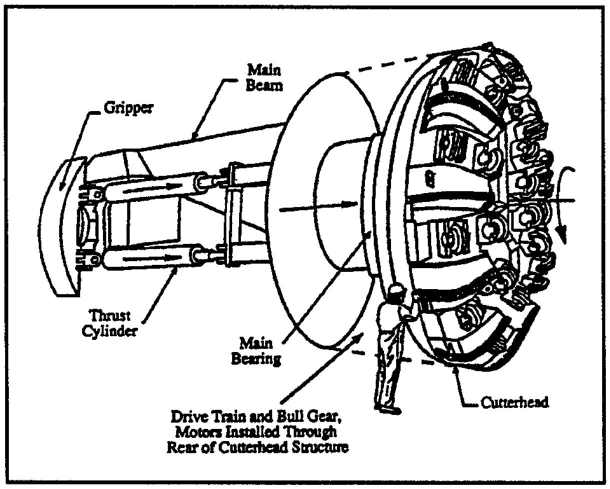


Figure 5-7. Unshielded TBM schematic drawing

used here, the shift time on a project is all working hours, including time set aside solely for maintenance purposes. All shift time on a project is therefore either mining time when the TBM operates or downtime when repairs and maintenance occur. Therefore,

$$Shift\ time = TBM\ mining\ time + Downtime$$

(b) *Penetration rate.* When the TBM is operating, a clock on the TBM will record all operating time. The TBM clock is activated by some minimum level of propel pressure and/or by a minimum torque and the start of cutterhead rotation. This operating time is used to calculate the penetration rate (*PR*), as a measure of the cutterhead advance per unit mining time.

Therefore,

$$PR = distance\ mined/TBM\ mining\ time$$

PR is often calculated as an average hourly value over a specified basis of time (i.e., instantaneous, hour, shift, day, month, year, or the entire project), and the basis for calculation should be clearly defined. When averaged over an hour or a shift, *PR* values can be on the order of 2 to 10 m per hour. The *PR* can also be calculated on the basis of distance mined per cutterhead revolution and expressed as an instantaneous penetration or as averaged over each thrust cylinder cycle or other time period listed above. The particular case of penetration per cutterhead revolution is useful for the study of the mechanics of rock cutting and is here given the notation PRev (penetration per revolution). Typical values of PRev can be 2 to 15 mm per revolution.

(c) *Utilization.* The percentage of shift time during which mining occurs is the Utilization, *U.*

$$U\ (\%) = TBM\ mining\ time/Shift\ Time \times 100$$

and is usually evaluated as an average over a specified time period. It is particularly important that *U* is reported together with the basis for calculation—whole project (including start-up), after start-up "production" average, or *U* over some other subset of the job. On a shift basis, *U* varies from nearly 100 percent to zero. When evaluated on a whole project basis, values of 35 to 50 percent are typical. There is no clear evidence that projects using a reconditioned machine have a lower *U* than projects completed with a new machine. Utilization depends more on rock quality, equipment condition, commitment to maintenance, contractor capabilities, project conditions (entry/access, alignment curves, surface space constraints on operations), and human factors (remoteness, underground temperature, and environment).

(d) *Advance rate (AR). AR* is defined on the basis of shift time as:

$$AR = Distance\ mined/Shift\ time$$

If *U* and *PR* are expressed on a common time basis, then the *AR* can be equated to:

$$AR = PR\ U\ (\%)/100$$

Advance rate can be varied by changes in either *PR* (such as encountering very hard rock or reduced torque capacity when TBM drive motors fail) or in *U* changes (such as encountering very poor rock, unstable invert causing train derailments, or highly abrasive rock that results in fast cutter wear).

(e) *Cutting rate (CR).* CR is defined as the volume of intact rock excavated per unit TBM mining time. Again, the averaging time unit must be defined clearly, and typical values of CR range from 20 to 200 m^3 per TBM mining hour.

(2) TBM performance from the UT database is summarized in Table 5-4. Other performance parameters deal

Table 5-4
TBM Performance Parameters for Projects in the UT Database

Parameter	Average	Range
Project length, km	3.80	0.1 - 36.0
Diameter, m	4.4	2.0 - 12.2
Advance rate, m/month	375	5 - 2,084
Advance rate, m/shift hr	1.2	0.3 - 3.6
Penetration rate m/TBM hr	3.3	0.6 - 8.5
Penetration rate mm/cutterhead revolution	7.2	1.0 - 17.0
Utilization, %	38	5 - 69

with evaluation of disc cutter replacement rate, which depends on cutter position and type of cutter, rock properties and also thrust, diameter, and cutterhead rotation rate. Parameters used to evaluate cutter replacement rates include average TBM mining time before replacement, linear distance of tunnel excavated per cutter change, distance rolled by a disc cutter before replacement (the rolling life), and rates of material wear from disc measurements (expressed as weight loss or diameter decrease). Rolling life distances for the replaceable steel disc edges may be 200 to 400 km for abrasive rock, to more than 2,000 km for nonabrasive rock, and is longer for larger diameter cutters. Appendix C contains information on TBM performance evaluation and cost estimating.

e. General considerations for TBM application. Important project features that indicate use of TBM include low grades (<3 percent preferable for tunnel mucking and groundwater management) and driving up hill. A minimum grade of 0.2 percent is required for gravity drainage of water inflow. Horizontal curves in an alignment can be negotiated by an open TBM with precision and little delay if curve radii are on the order of 40 to 80 m. Most shielded TBMs and back-up systems are less flexible, however, so that a minimum radius of 150 to 400 m should generally be used for design purposes. Tighter curves should be avoided or planned in conjunction with a shaft to facilitate equipment positioning.

(1) Experience indicates that tunnel depth has little impact on advance rates in civil projects, providing that the contractor has installed adequate mucking capacity for no-delay operation. Therefore, tunnel depth should be chosen primarily by location of good rock. Portal access, as opposed to shafts, will facilitate mucking and material supply, but more important is that the staging area for either shaft or portal be adequate for contractor staging. Confined surface space can have a severe impact on project schedule and costs. For long tunnels, intermediate access points can be considered for ventilation and mucking exits. However, assuming the contractor has made appropriate plans for the project, a lack of intermediate access may not have a significant impact on project schedule.

(2) In planning a project schedule, the lead time needed to get a TBM onsite varies from perhaps 9 to 12 months for a new machine from the time of order, to perhaps 3 to 6 months for a refurbished machine, and to nearly no time required for a direct re-use without significant repairs or maintenance. With proper maintenance, used TBMs can be applied reliably, and there is little need to consider specifying new equipment for a particular project. TBM cutterheads can be redesigned to cut excavated diameters different by 1 to 2 m, but the thrust and torque systems should also be modified accordingly.

(3) With delivery of a TBM onsite, about 3 to 6 weeks will be required for assembly, during which time a starter tunnel should be completed. The start of mining rarely occurs with the full back-up system in place. Decreased advance rates on the order of 50 percent less than for production mining should be expected for the first 4 to 8 weeks of mining, as the back-up system is installed and the crew learns the ropes of system operation.

f. Specification options for TBMs. Specifications can be either prescriptive or performance specifications. If specifications include prescriptive information on performing work and also specific standards to be achieved in the finished product, disputes are likely. Make sure all specification provisions are compatible with provisions in the GDSR. If there are discrepancies or ambiguities, disputes can be expected.

(1) *New versus reconditioned equipment.* There is no statistically significant difference in performance between new and reconditioned equipment. Leaving the option open for contractors will tend to decrease costs. Exceptions include very long tunnels for which major equipment downtime for main-bearing repairs would be disastrous and hazardous ground conditions for which special TBM capabilities are required. Rebuilds are possible to ±10 percent of the original TBM diameter, but consideration should be given to the need to upgrade the thrust and torque systems if TBM diameter is increased, particularly if there is a significant difference in the rock between the previous and

current project. A given TBM may perform acceptably in weaker rock, but may be underpowered for harder rock.

(2) *Level of detail in specifications.* The key here is to specify only what is required by the designer for success in mining and support. Performance specifications are preferable. Reasonable specification requirements might include the following:

(a) Expected short stand-up time where support installation must be rapidly placed.

(b) Squeezing ground conditions with which the shield must be able to cope.

(c) Adequate groundwater handling system capacity onsite.

(d) Special equipment, safety management, and special operating procedures for gassy ground.

(e) Expectations for the contractor to supply a TBM capable of a minimum PRev, and a back-up system sized to provide no-delay mucking.

(3) *Contractor submittals.* The designer should ask for only what is important and what he or she is prepared to review. For example, an engineer could ask the contractor to demonstrate that the mucking system capacity will be adequate to support no-delay mining, or the contractor might be asked for information on time to install support if stand-up time is expected to be critical to the mining operation.

g. Record keeping and construction monitoring. During construction, it is very important that the resident staff gather information concerning the progress of construction and the encountered ground conditions. Such information is paramount to understand and document any changing ground conditions and to evaluate the impact of changing conditions on the operations of the contractor and vice versa. The information important to monitoring TBM construction include the following:

(1) Shift records of contractor activities should be maintained throughout a contract, but primarily at the heading. Shift reports should include the following information:

(a) Sequential time log of each shift including all activities.

(b) Downtime including reasons for shutdown.

(c) Record of thrust and torque (motor-operating amperage, number of motors on line, cutterhead rotation rate, thrust pressure, and gripper pressure), tunnel station, and TBM clock time elapsed for each stroke cycle of mining.

(d) Record of all cutters changed, including TBM clock time and station for each replacement, disc position on the cutterhead and reason for replacement (such as disc wear, bad bearing, split disc, etc.).

(e) Start and end station for each shift and for each stroke cycle.

(f) Information on ground conditions, groundwater encountered, and support installed, identified by station.

(g) Information on survey/alignment control and start/end of alignment curves.

(2) Records of installed support should be maintained in detail by the resident engineer. These can be incorporated in the tunnel geologic maps.

(3) Maps of as-encountered geologic conditions should be made for all tunnels driven with open TBMs. For shielded TBMs, all opportunities to view the rock at the heading should be mapped. The site geologist should maintain maps of the tunnel walls and changing ground conditions together with an assessment of rock mass quality and should continue to compare mapped information with predictions made at the time of site investigation and update or anticipate any notable systematic changes.

5-4. Initial Ground Support

a. General

Initial ground support is usually installed concurrently with the excavation. For drill and blast excavations, initial ground support is usually installed after the round is shot and mucked out and before drilling, loading, and blasting of the next round. For TBM-driven tunnels, excavation is carried out more or less continuously, with the support installed as the TBM moves forward. Because of the close relationship between excavation and initial support activities, they must be well coordinated and should be devised such that the process is cyclic and routine. Initial ground support may consist of steel ribs, lattice girders, shotcrete, rock dowels, steel mesh, and mine straps. The main purposes served by these support elements include stabilizing and preserving the tunnel after excavation and providing worker safety. As the quality of the rock increases, the

amount of required initial ground support decreases. After installation of initial ground support, no other additional support may be required. In this case, the initial support will also fulfill the role of final support. In other cases, additional support, such as a cast-in-place concrete lining, may be installed. The initial and the final ground support then comprise a composite support system. An example of tunnel support fulfilling the initial and final support functions is when precast concrete segmental linings are used to support a tunnel in weak rock behind a TBM. One issue that must be considered when contemplating the use of initial support for final support is the longevity of the initial support components. While these components may behave satisfactorily in the short term, phenomena such as corrosion and deformation must be considered for permanent applications.

b. Initial ground reinforcement. Initial ground reinforcement consists of untensioned rock dowels and, occasionally, tensioned rock bolts. These are referred to as ground reinforcement, because their function is to help the rock mass support itself and mobilize the inherent strength of the rock as opposed to supporting the full load of the rock. It is much more economical to reinforce the rock mass than to support it. The reinforcement elements are installed inside the rock mass and become part of the rock mass. Rock support such as concrete linings and steel sets restrict the movements of the rock mass and offer external support to the rock mass. The design and construction of rock reinforcement systems are discussed in EM 1110-1-2907. The subject is addressed herein only as it relates to the construction of tunnels. There are three types of rock bolts (Stillborg 1986):

- Mechanically anchored (rock bolts) (Figure 5-8).
- Grouted bars (dowels) (Figures 5-9 and 5-10).
- Friction dowels (Figures 5-11 and 5-12).

Friction dowels are usually considered temporary reinforcement because their long-term corrosion resistance is uncertain. Typical technical data on these types of rock bolts and dowels are given in Table 5-5.

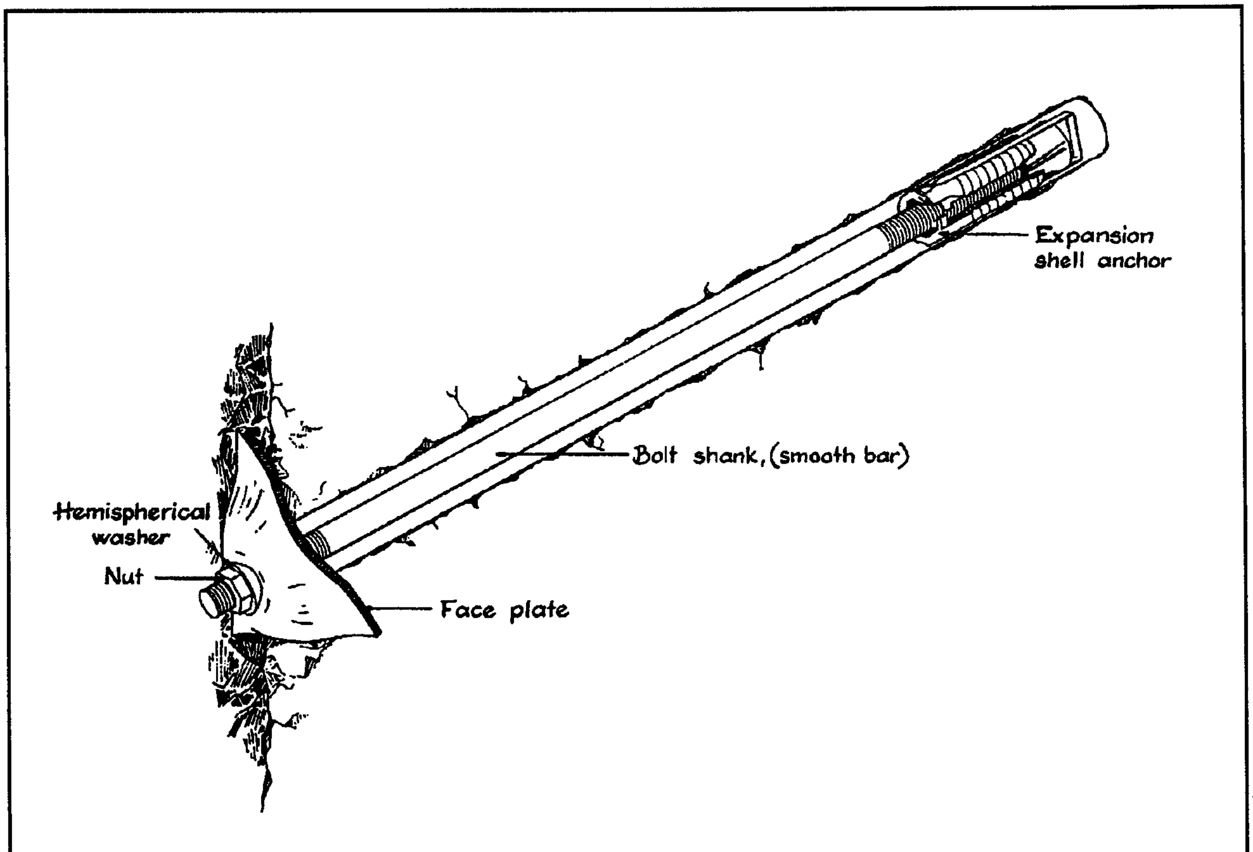


Figure 5-8. Mechanically anchored rock bolt–expansion shell anchor

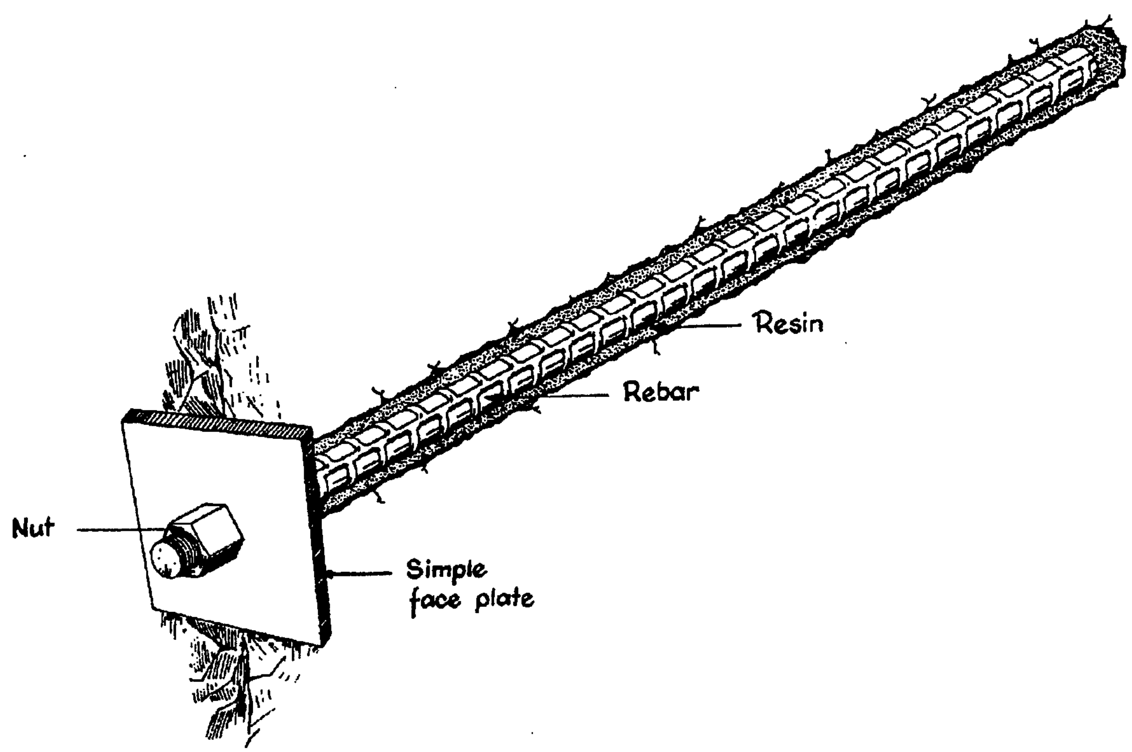


Figure 5-9. Grouted dowel—rebar

(1) *Installation.* To install a rock bolt or dowel, a borehole must be drilled into the rock of a specific diameter and length no matter what type of bolt or dowel is being used. This can be accomplished with a jack leg for small installations or a drill jumbo when high productivity is required. Special rock dowel installation gear is often used. In a blasted tunnel, the drill jumbo used for drilling the blast holes is frequently used to drill the rock bolt holes. Except for split sets, the diameter of the rock bolt hole can vary somewhat. It is common to have up to 10 or 20 percent variation in the hole diameter because of movement and vibration of the drill steel during drilling and variations in the rock. For expansion anchors and grouted and Swellex bolts, this is not a serious problem. Split sets are designed for a specific diameter hole, however; if the hole is larger, it will not have the required frictional resistance. Therefore, drilling of the hole for split sets must be closely controlled. After the hole is drilled, it should be cleaned out (usually with an air jet) and the bolt or dowel installed promptly. For mechanically anchored rock bolts, the bolt is preassembled, slid into the hole, and tightened with a torque wrench. The final tension in the bolt should be created by a direct-pull jack, not by a torque wrench. For resin-grouted rock dowels, the grout is placed in the hole using premade two-component cartridges; the bar is installed using a drill that turns the bar, breaks open the cartridges, and mixes the two components of the resin. The time and method of mixing recommended by the manufacturer should be used. Cement-grouted dowels can be installed the same way except that the grout is pumped into the hole through a tube in the center of the bar.

(2) *Tensioning.* Grouted bolts can be left untensioned after installation or can be tensioned using a torque wrench or a hydraulic jack (Figure 5-13) after the grout has reached adequate strength. Fast-set resin grout can be used to hasten the process for resin-grouted bolts. Cement grout takes longer to cure even if an accelerator is used. Rock bolts in tunnels are usually left untensioned after installation and become tensioned as the rock mass adjusts to the changes in stress brought on by the process of excavation.

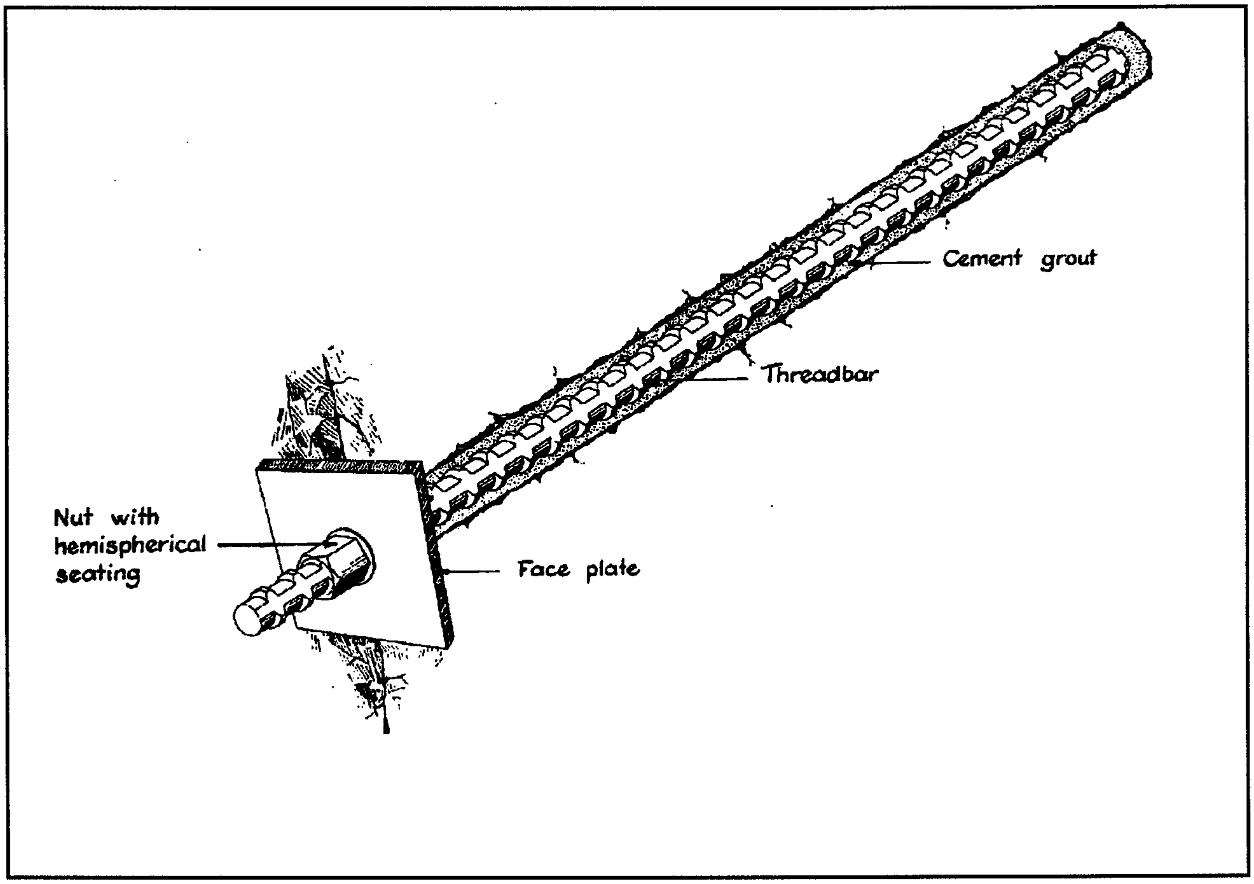


Figure 5-10. Grouted dowel—Dywidage ® Steel

Split sets and Swellex bolts work this way since they cannot be pretensioned. There are cases when pretensioning the bolt is necessary, such as to increase the normal force across a joint along which a wedge or block can slip.

(3) *Hardware*. Rock bolts usually have end plates (Figure 5-14) held in place with nuts and washers on the ends of bars or by enlargement of the head of split sets and Swellex bolts. End plates provide the reaction against the rock for tensioned bolts. End plates also are used to hold in place steel mesh and mine straps. They can also be embedded in shotcrete to provide an integral system of rock reinforcement and surface protection (Figure 5-15). End plates are generally square, round, or triangular shaped (Figure 5-16). Steel mesh, mine straps, and shotcrete are used to hold small pieces of rock in place between the rock bolts.

(4) *Testing*. Testing rock bolts is an important part of the construction process. If the rock bolts are not adequately installed, they will not perform the intended function. Possible reasons for faulty installations include the following:

- Incorrect selection of the rock bolt system.
- Incorrect placement of borehole.
- Incorrect length of borehole.
- Incorrect diameter of borehole.
- Inadequate cleaning of borehole.
- Inadequate placement of grout.
- Inadequate bond length of grout.
- Corrosion or foreign material on steel.
- Misalignment of rock bolt nut and bearing plate assembly.

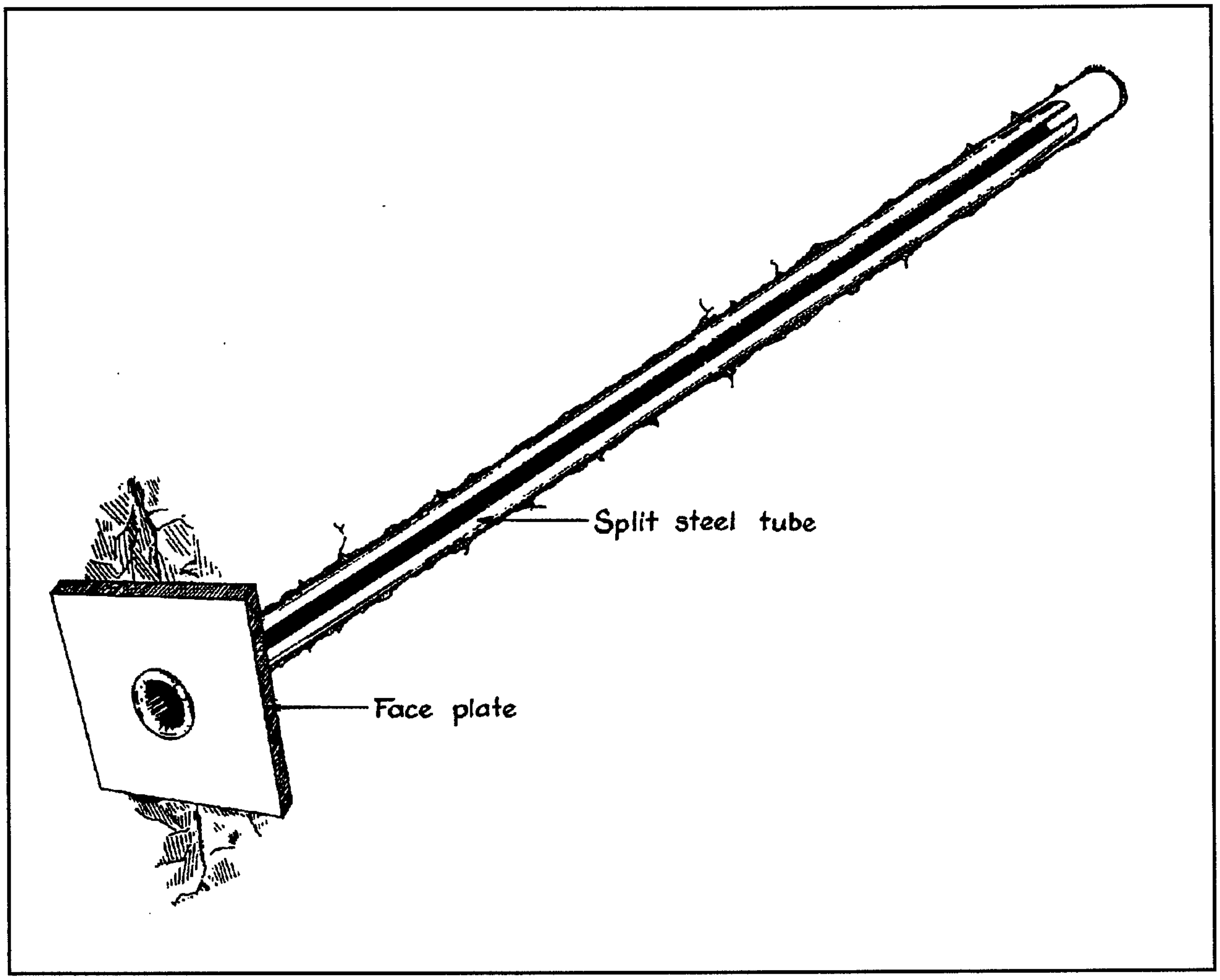


Figure 5-11. Friction dowel—Split Set ®

- Out-of-date grouting agents.
- Inappropriate grout mixture.
- Damage to breather tube.
- Inadequate borehole sealing.
- Inadequate lubrication of end hardware.
- Incorrect anchor installation procedure.
- Inadequate test program.
- No monitoring of rock bolt system performance.

Many of these problems can be avoided by adherence to manufacturer installation recommendations, and manufacturer representatives may be required to be onsite at the beginning of rock bolting operations to ensure conformance and trouble-shoot problems. The most common method of testing rock bolts or dowels is the pull-out test. A hydraulic jack is attached to the end of the rock bolt and is used to load the rock bolt to a predetermined tensile load and displacement. Rock bolts may be tested to failure or to a lesser value so that they can be left in place to perform their intended function. If the test load or displacement is exceeded, that rock bolt or dowel has failed and others in the area are tested to see if the failure is an isolated problem or indicative of a systematic problem related to all of the bolts or dowels. Usually, many units are tested at the

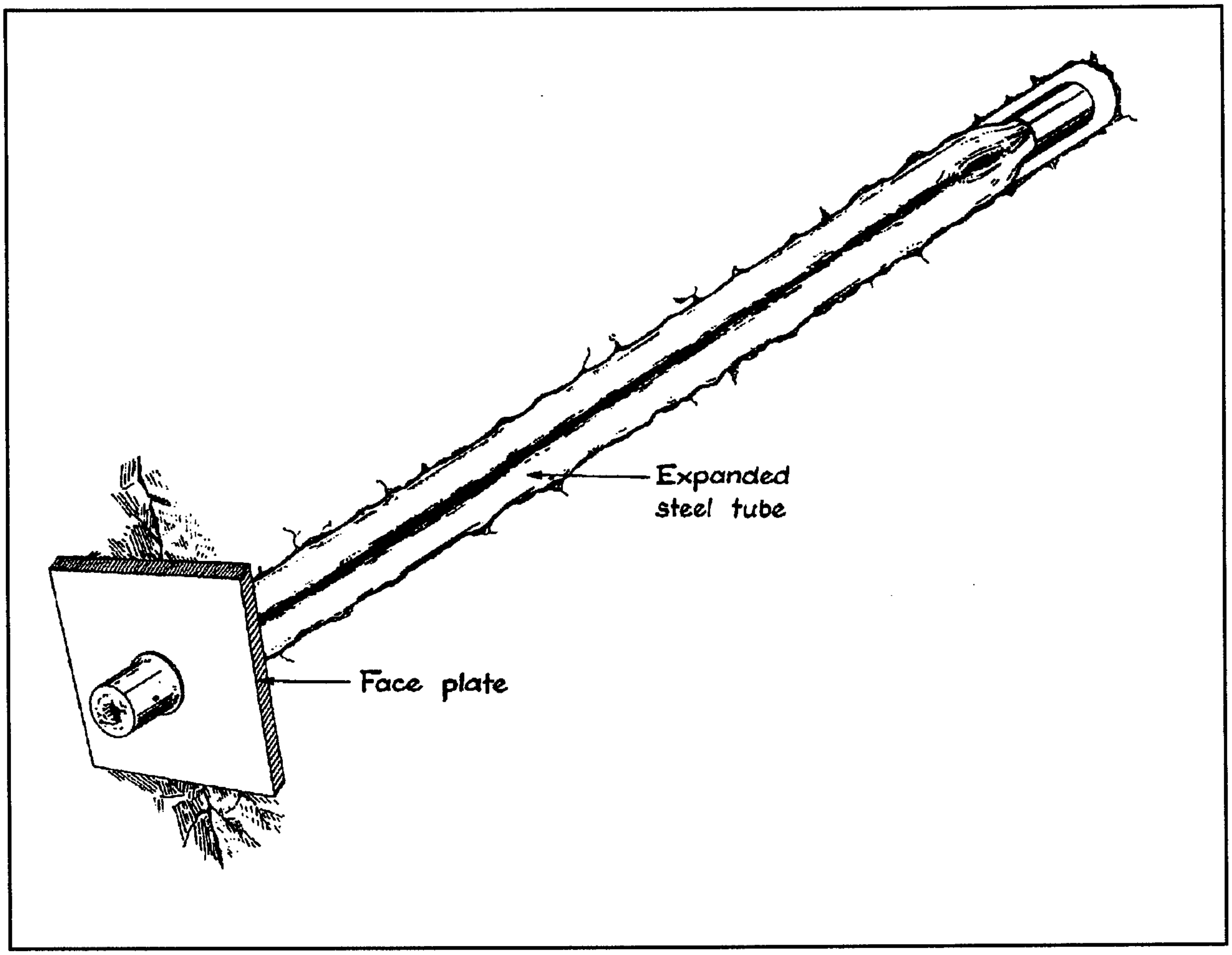


Figure 5-12. Friction dowel—Swellex ®

beginning of tunneling, and once installation procedures, methods, and personnel skills are adequately confirmed, then a more moderate testing rate is adopted. If problems occur, changes are made, and a more rigorous testing scheme is reinstated until confidence is restored. Pull-out tests do not test the entire dowel. Only that length of the dowel that is required to resist the pull-out force is tested. For example, a dowel may be only partially grouted and still resist the pull-out force. These uncertainties are generally accepted in tunnel construction, and credence is placed on tunnel performance and pull-out test results. To further test the installation, the dowel can be overcored and exhumed from the rock for direct inspection. However, this requires costly special equipment and is only done under unusual circumstances. Other methods of testing include checking the tightness of a mechanically anchored rock bolt with a torque wrench, installing load cells on the end of tensioned rock bolts, and nondestructive testing by transmitting stress waves down through the bolt from the outer end and monitoring the stress wave return. The less stress wave reflection that is observed, the better the installation is. Swellex bolts can be tested using nondestructive techniques by reattaching the installation pump to the end of the bolt and testing to see that the tube still holds the same amount of pressure as when it was installed.

c. Shotcrete application. Shotcrete today plays a vital role in most tunnel and shaft construction in rock because of its versatility, adaptability, and economy. Desirable characteristics of shotcrete include its ability to be applied immediately to freshly excavated rock surfaces and to complex shapes such as shaft and tunnel intersections, enlargements, crossovers, and bifurcations and the ability to have the applied thickness and mix formulation varied to suit variations in ground behavior. A brittle

Table 5-5
Typical Technical Data on Various Rock Bolt Systems

Item	Mechanically Anchored	Resin-Grouted Bolts (Rebar)	Cement-Grouted Bolts (Dywidag)	Friction Anchored (Split Set)	Friction Anchored (Swellex)
Steel quality, MPa	700	570	1,080	Special	Special
Steel diameter, mm	16	20	20	39	26
Yield load, steel, kN	140	120	283	90	130
Ultimate load, steel kN	180	180	339	110	130
Ultimate axial strain, steel, %	14	15	9.5	16	10
Weight of bolt steel kg/m	2	2.6	2.6	1.8	2
Bolt lengths, m	Any	Any	Any	0.9-3	Any
Usual borehole diameter, mm	35-38	30-40	32-38	35-38	32-38
Advantages	Inexpensive. Immediate support. Can be permanent. High bolt loads.	Rapid support. Can be tensioned. High corrosion resistance. Can be used in most rocks.	Competent and durable. High corrosion resistance. Can be used in most rocks. Inexpensive.	Rapid and simple installation. Immediate support. No special equipment.	Rapid and simple installation. Immediate support. Good for variety of conditions.
Disadvantages	Use only in hard rock. Difficult to install reliably. Must check for proper tensioning. Can loosen due to blasting.	Messy. Grout has limited shelf life. Sensitive to tunnel environment.	Takes longer to install than resin bolts. Can attain high bolt loads.	Expensive. Borehole diameter crucial. Only short lengths. Not resistant to corrosion.	Expensive. Not resistant to corrosion. Special pump required.

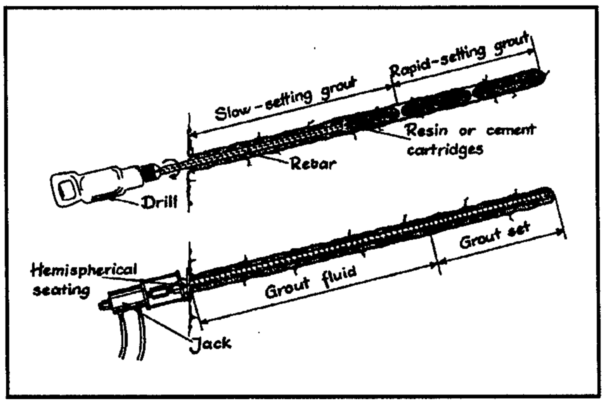


Figure 5-13. Tension resin dowel installation

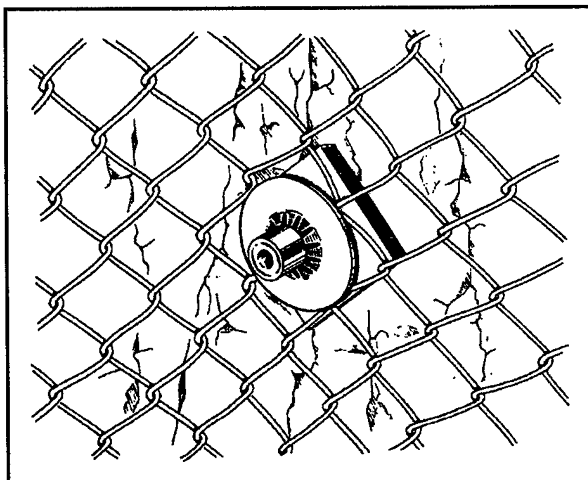


Figure 5-14. Mesh washer end hardware

material by nature, shotcrete used for ground support often requires reinforcement to give it strain capacity in tension (i.e., ductility) and to give it toughness. Chain link mesh or welded wire fabric has long served as the method to reinforce shotcrete, but has now been largely supplanted by steel fibers mixed with the cement and the aggregate. Steel fiber reinforced shotcrete (SFRS) was first used in tunnels in North America by the USACE in 1972 in an adit at Ririe Dam (Idaho) (Morgan 1991). In addition to improving toughness and flexural strength, steel fibers improve the fatigue and impact resistance of the shotcrete

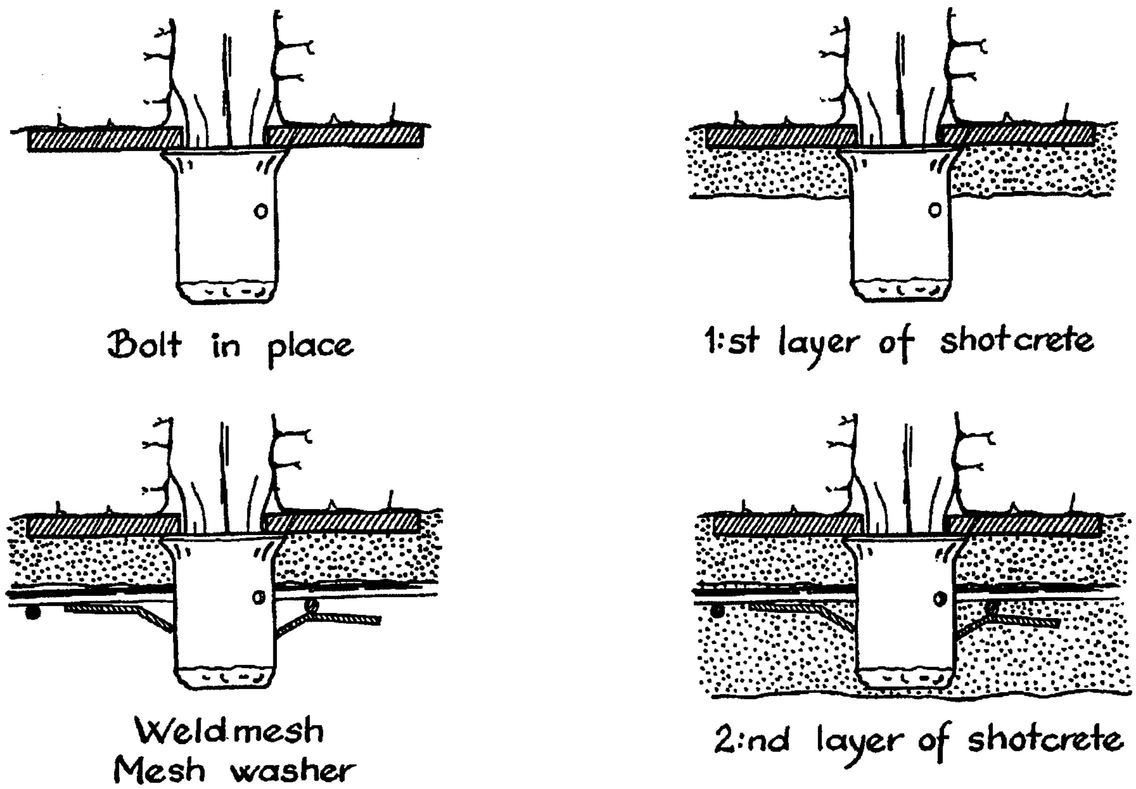


Figure 5-15. Dowels with end hardware embedded in shotcrete

layer. Other relatively recent improvements to shotcrete applications include admixtures for a variety of purposes, notable among which is the use of microsilica, which greatly reduces rebound and increases density, strength, and water tightness. EM 1110-2-2005 provides guidance in the design and application of shotcrete.

(1) *Range of applications.* For most tunnels and shafts, shotcrete is used as an initial ground support component. It is sprayed on freshly exposed rock in layers 2 to 4 in. thick where it sets in a matter of minutes or hours, depending on the amount of accelerator applied, and helps support the rock. In blasted rock with irregular surfaces, shotcrete accumulates to greater thicknesses in the overbreaks. This helps prevent block motion and fallout due to shear, by adhering to the irregular surface. On more uniform surfaces, the shotcrete supports blocks by a combination of shear, adhesion, and moment resistance and supports uniform and nonuniform radial loads by shell action and adhesion. By helping prevent the initiation of rock falls, shotcrete also prevents loosening of the rock mass and the potential for raveling failure. Shotcrete also protects surfaces of rock types that are sensitive to changes of moisture content, such as swelling or slaking rock. The application of shotcrete is an essential ingredient in the construction method of sequential excavation and support, where it is used in combination with rock bolts or dowels and, sometimes, steel ribs or lattice girders in poor ground. For TBM tunnels, initial ground support usually consists of dowels, mesh, mine straps, channels, or steel ribs; shotcrete can be applied some distance behind the advancing face. Only in a few instances have TBMs been built with the possibility to apply shotcrete a short distance behind the face.

(2) *Reinforced shotcrete.* In poor or squeezing ground, additional ductility of the shotcrete is desirable.

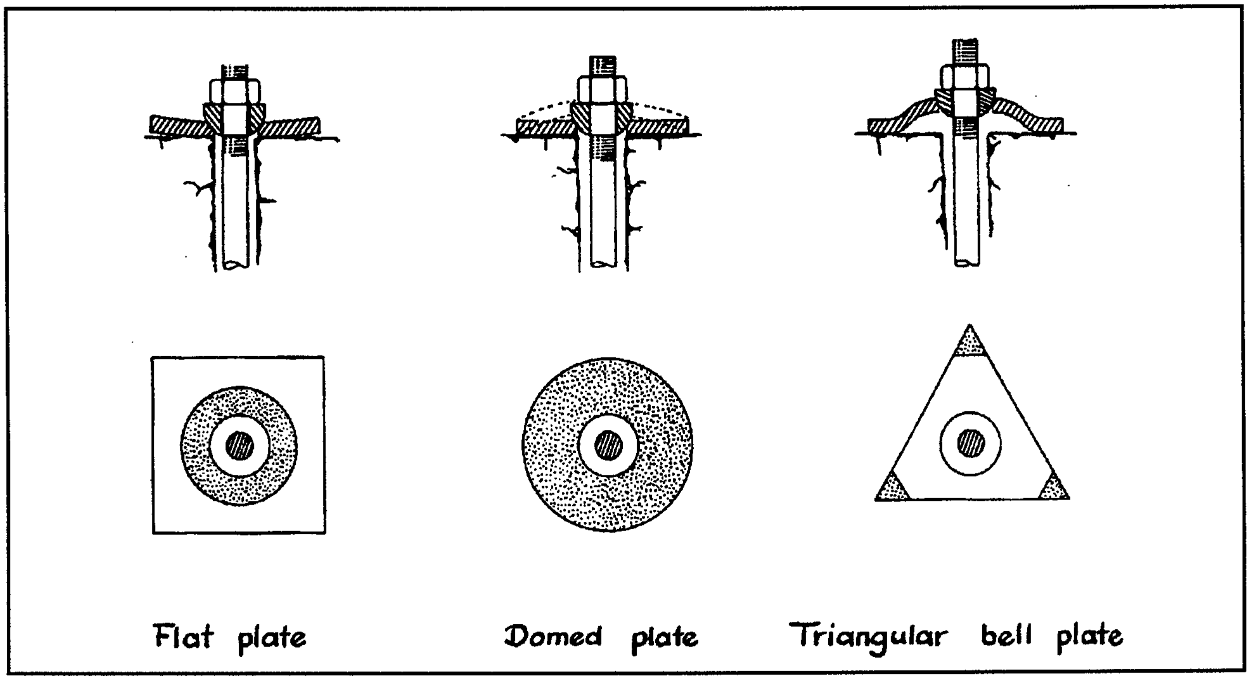


Figure 5-16. Different types of end hardware

Until recently this ductility was generally achieved by welded wire fabric usually applied between the first and the second coat of shotcrete. While wire fabric does add to the ductility of the shotcrete, it has several disadvantages. It is laborious and costly to place; it is difficult to obtain good shotcrete quality around and behind wires; and it often results in greater required shotcrete volumes, because the fabric cannot be draped close to the rock surface on irregularly shot surfaces. Modern reinforced shotcrete is almost always steel fiber-reinforced shotcrete. The steel fibers are generally 25- to 38-mm-long deformed steel strips or pins, with an aspect ratio, length to width or thickness, between 50 and 70. These steel fibers are added to the shotcrete mix at a rate of 50-80 kg/m^3 (85-135 lb/yd^3) without any other change to the mix. The steel fibers increase the flexural and tensile strength but more importantly greatly enhance the postfailure ductility of the shotcrete. Steel fibers are made and tested according to ASTM A 820 and steel fiber shotcrete according to ASTM C 1116.

d. Steel ribs and lattice girders. Installing steel and wooden supports in a tunnel is one of the oldest methods in use. Many years ago, wooden supports were used exclusively for tunnel support. In later years, steel ribs (Figure 5-17) took the place of wood, and, most recently, steel lattice girders (Figure 5-18) are being used in conjunction with shotcrete. Figure 5-19 shows an application of shotcrete, lattice girders, and dowels for a rapid transit tunnel through a fault zone. It is usually faster and more economical to reinforce the rock with rock bolts, steel mesh or straps, and shotcrete so the rock will support itself. However, if the anticipated rock loads are too great, such as in faulted or weathered ground, steel supports may be required. Steel ribs and lattice girders usually are installed in the tunnel in sections within one rib spacing of the tunnel face. The ribs are generally assembled from the bottom up making certain that the rib has adequate footing and lateral rigidity. Lateral spacer rods (collar braces) are usually placed between ribs to assist in the installation and provide continuity between ribs. During and after the rib is erected, it is blocked into place with grout-inflated sacks as lagging, or shotcrete. In modern tunnel practice, the use of wood blocking is discouraged because it is deformable and can deteriorate with time. The rib functions as an arch, and it must be confined properly around the perimeter. The manufacturer of steel ribs provides recommendations concerning the spacing of blocking points that should be followed closely (see Proctor and White 1946). When shotcrete is used as lagging, it is important to make sure that no voids or laminations are occurring as the shotcrete spray hits the steel elements. Steel ribs should be fully embedded in the shotcrete. The lattice girders are filled in by shotcrete in addition to being embedded in shotcrete. Steel ribs and lattice girders are often not the

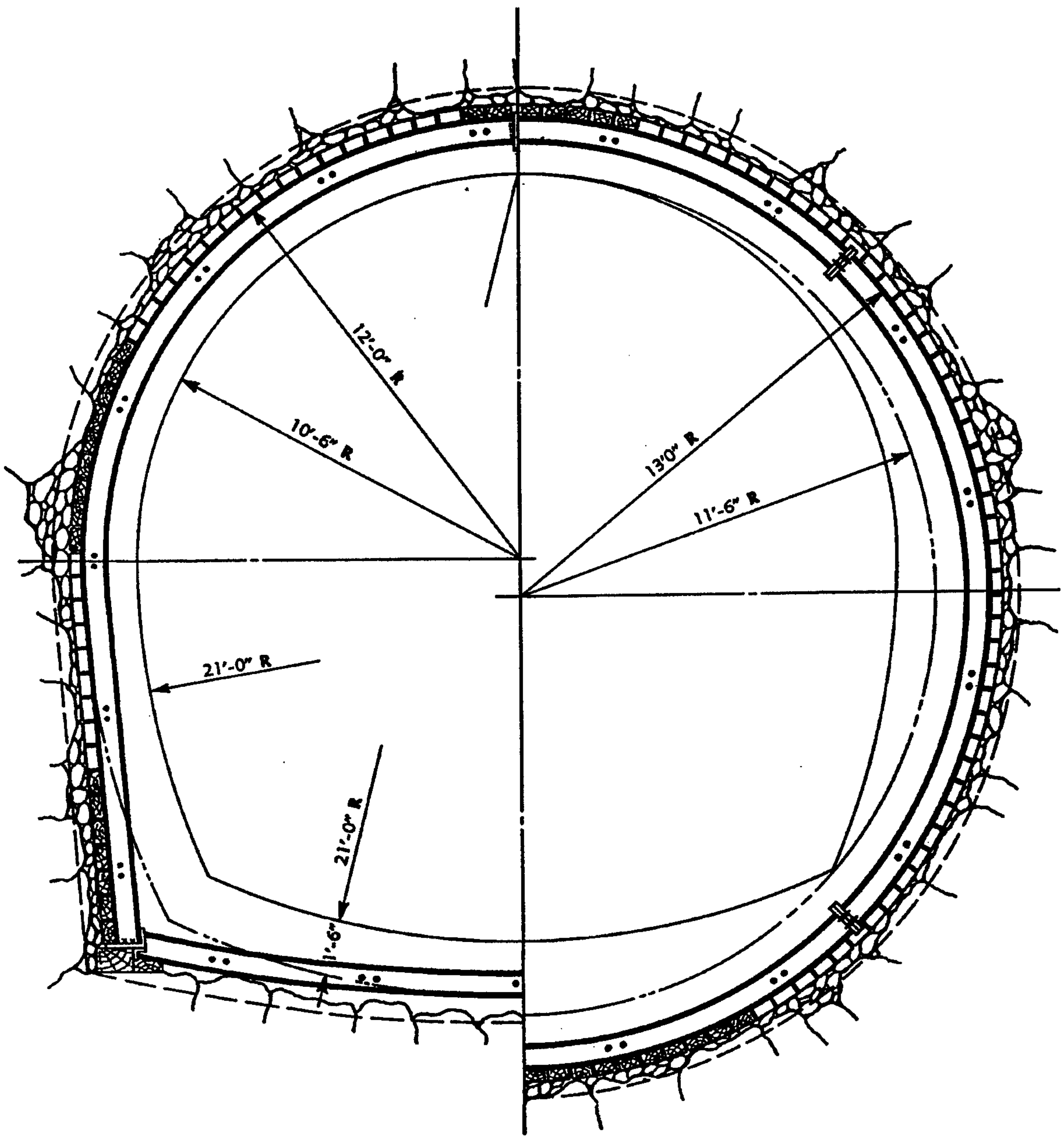


Figure 5-17. Steel rib examples, conversion of a horseshoe-shaped flow tunnel to a circular shape in squeezing ground

sole method of tunnel support but are only provided in the event that bad tunneling conditions are encountered. In this case, it is necessary to have all the required pieces at the site and have adequately trained personnel ready when they are required in order to reduce delays in switching to a different type of tunnel support.

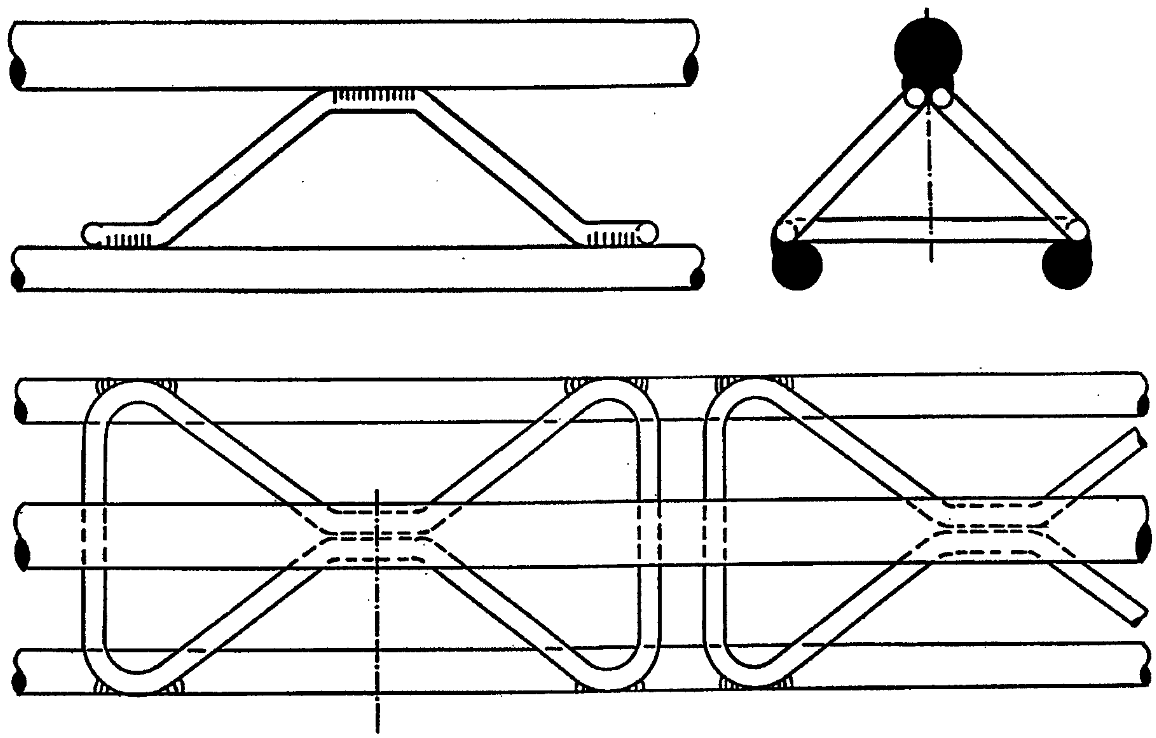


Figure 5-18. Lattice girders

e. Precast concrete segments used with TBM. Soft ground tunnels in the United States are most often constructed using shields or shielded TBMs with precast concrete segments. Below the groundwater table, the segments are bolted with gaskets for water tightness. Above the groundwater table, unbolted, expanded segmental linings are often used, followed by a cast-in-place concrete lining (two-pass lining). If necessary, a water- or gasproofing membrane is placed before the cast-in-place concrete is placed. The shield or TBM is usually moved forward using jacks pushing on the erected segmental concrete lining. Hard rock tunnels driven with a TBM may also be driven with some form of segmental lining, either a one-pass or two-pass lining. There are several reasons for this choice.

(1) For the completion of a long tunnel, the schedule may not permit the length of time required to cast a lining in place. The option of casting lining concrete while advancing the TBM is feasible, at least for a large-diameter tunnel, but often not practical. Interference between concrete transportation and placement and tunnel excavation and mucking is likely to slow tunnel driving. Transporting fresh concrete for a long distance can also be difficult. In this instance, placing a one-pass segmental lining is a practical solution, provided that lining erection does not significantly slow the advance of the TBM.

f. Bolted or unbolted segments. A gasketed and bolted segmental lining must be fabricated with great precision, and bolting extends the time required for erection. Hence, such a lining is usually expensive to manufacture and to erect. For most water tunnels, and for many other tunnels, a fully gasketed and bolted, watertight lining is not required, and an unbolted segmental lining is adequate.

g. Segment details. Once a segmental lining has been determined to be feasible or desirable, the designer has a number of choices to make. In the end, the contractor may propose a different lining system of equal quality that better fits his/her proposed methods of installation. A

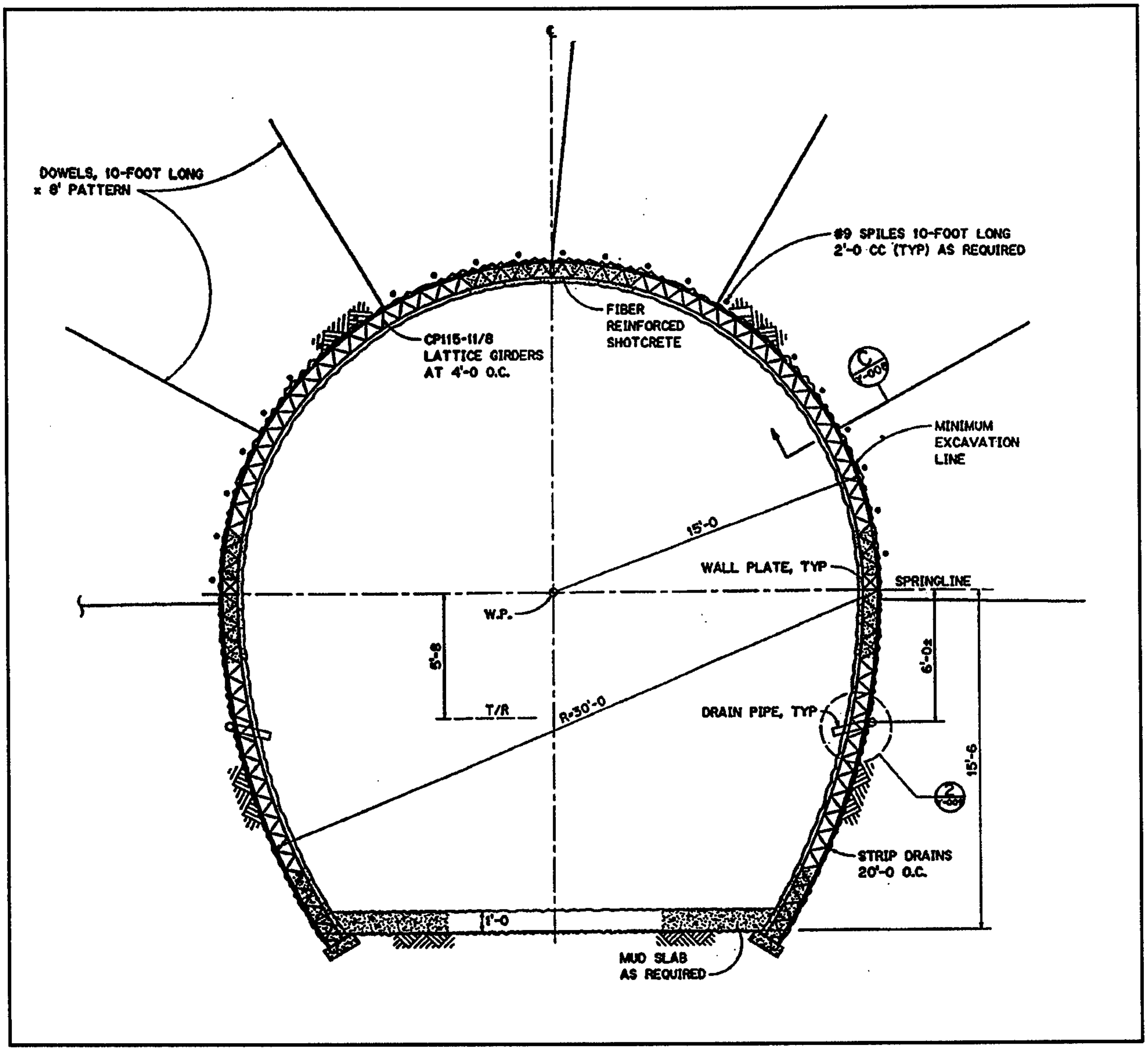


Figure 5-19. Lattice girders used as final support with steel-reinforced shotcrete, dowels, and spiles

selection of lining and joint details are shown on Figures 5-20 to 5-22. Details are selected to meet functional requirements, and for practicality and economy of construction. For the most part, details can be mixed liberally to match given requirements and personal preferences.

h. Matching construction methods and equipment. When a tunnel lining system has been selected, construction methods and equipment must be designed to match the specific needs of this system. With a full shield tail, the invert segment is placed on the shield surface at the bottom. When the shield passes, the invert segment falls to the bottom, unless it is bolted to the previous segment. The erector equipment must match the pick-up holes in the segments, be able to rotate the segment into its proper place, and must have all of the motions (radial, tangential, axial, tilt, etc.) to place the segment with the tolerances required. Relatively high speed motion is required to bring a segment to its approximate location, but inching speed is often required for precise positioning. Unless each segment is stable as placed, holding devices are required to prevent them from falling out until the last segment is in place. Such holding devices are not required for a bolted and for most dowelled linings.

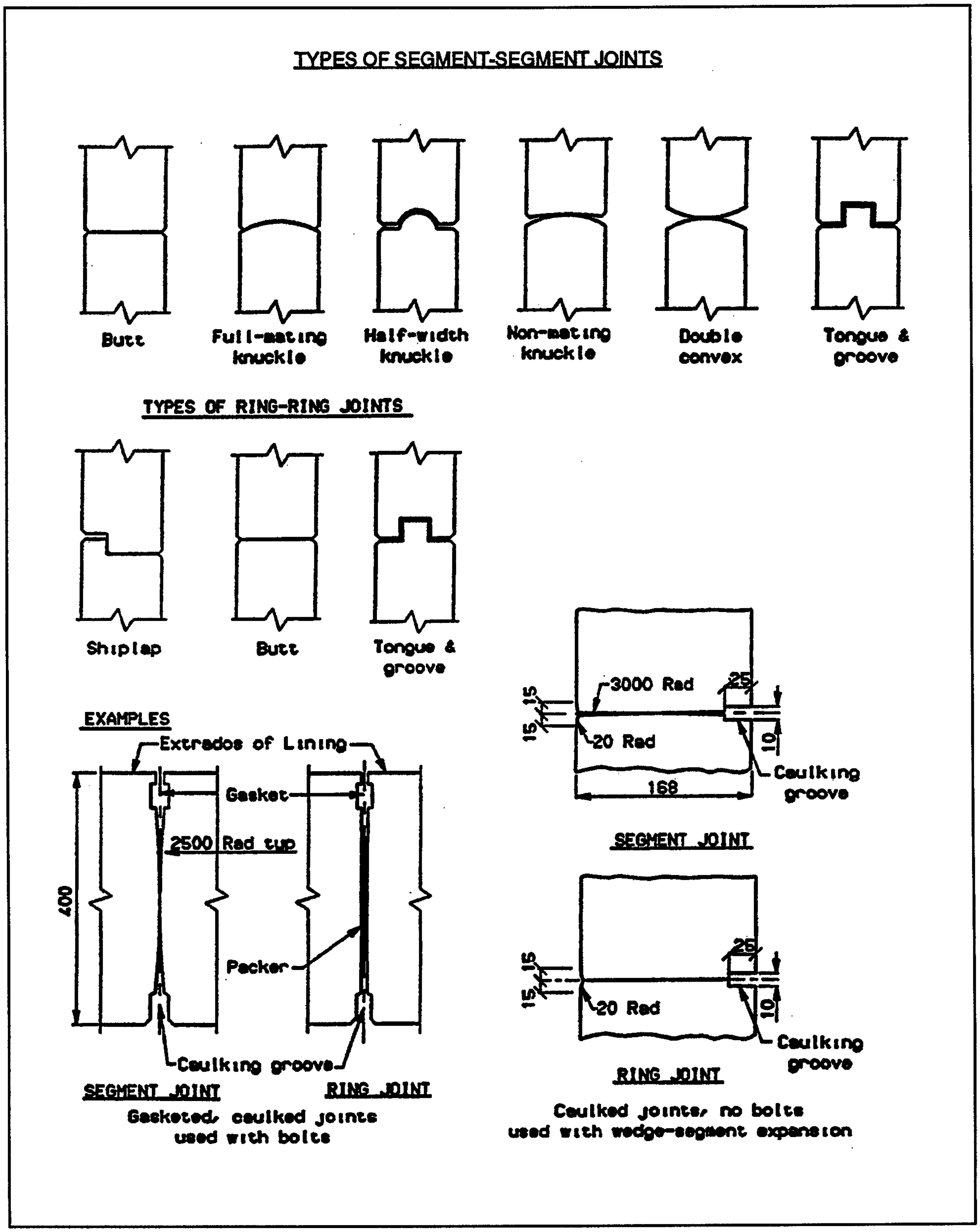


Figure 5-20. Types of joints in segmental concrete lining

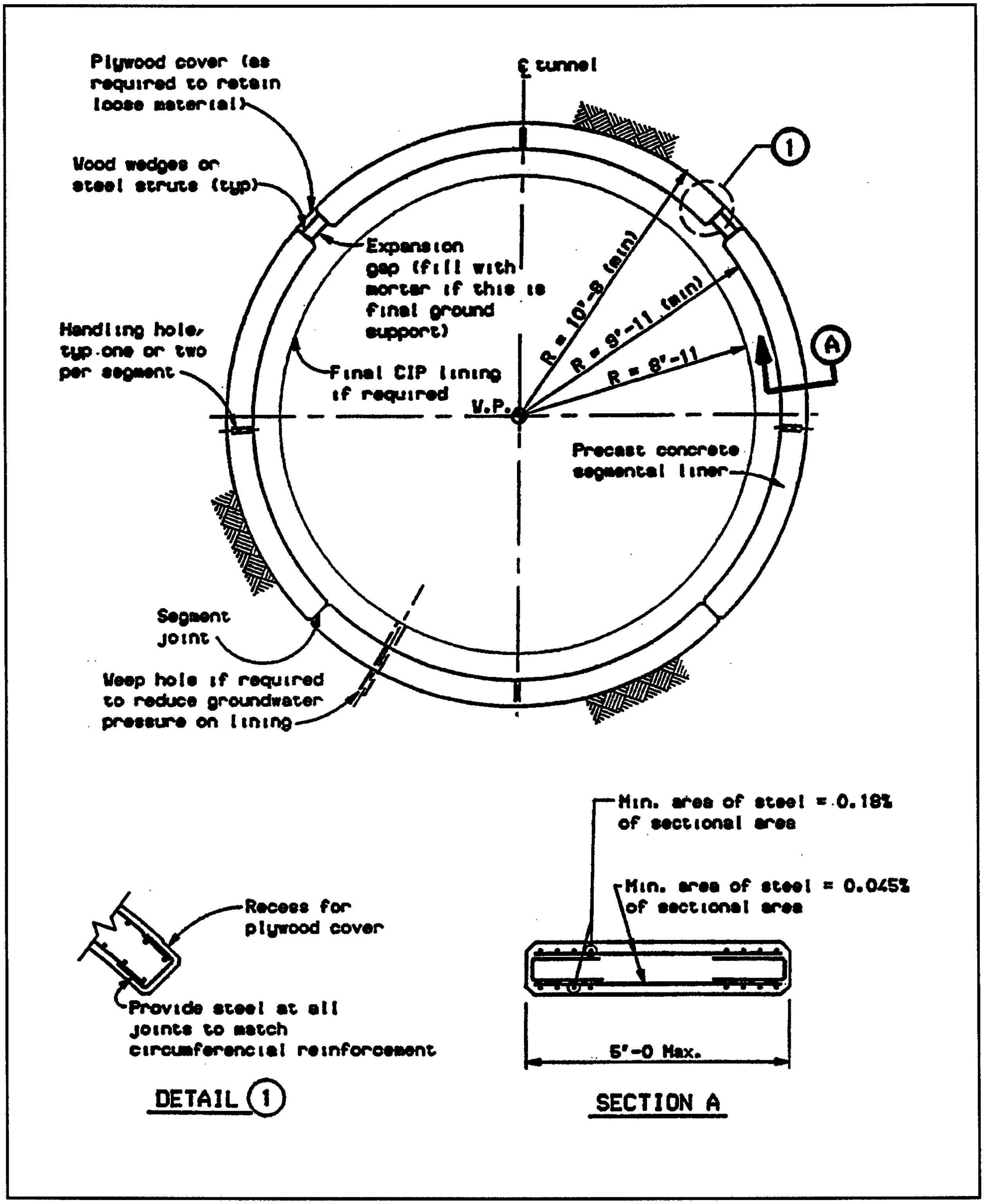


Figure 5-21. Simple expanded precast concrete lining used as initial ground support or as final ground support

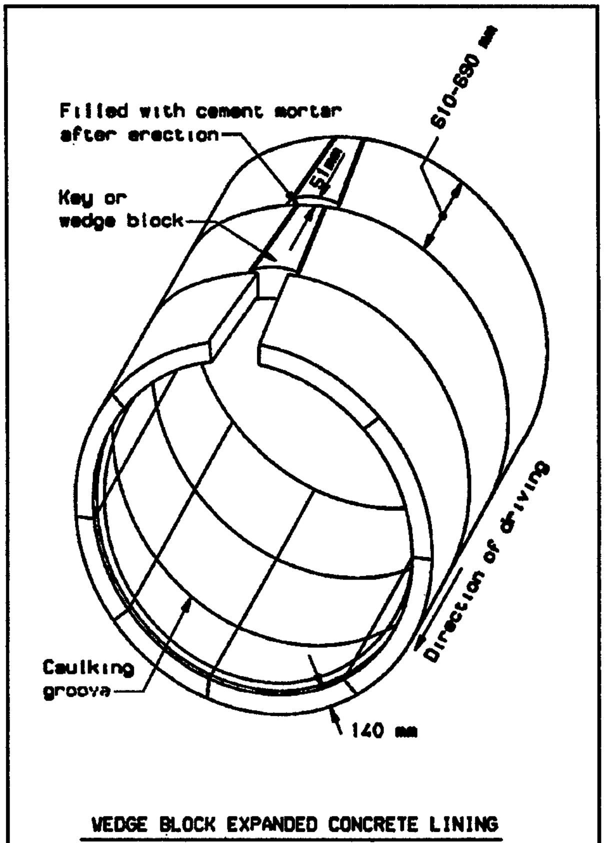


Figure 5-22. Wedge block expanded concrete lining

i. Functional criteria for one-pass segmental linings.

(1) Selection of a segmental lining system is based on considerations of cost and constructibility, and many details depend on the construction procedure. Functional criteria, however, must also be met.

(2) Water flow and velocity criteria often require a smooth lining to achieve a reasonably low Mannings number. This may require limitations on the offset permitted between adjacent segment rings. With an expanded lining, it is often not possible to obtain full expansion of all rings, and offsets between rings can be several centimeters. If this is not acceptable, an unexpanded dowelled or bolted ring may be required.

(3) In the event that some segments are, in fact, erected with unacceptable offsets, the hydraulic effect can be minimized by grinding down the protrusions or filling the shadows.

(4) A watertight lining is difficult to obtain using segments without gaskets. In some lining systems, sealing strips or caulking are employed to retain grout filling, but cannot sustain high groundwater pressures. In wet ground, it may be necessary to perform formation grouting to reduce water flows. Alternatively, fully gasketed and bolted linings may be used through the wet zones. This choice depends on the acceptability of water into or out from the tunnel during operations and the differential water pressure between the formation and the tunnel. The choice also depends on the practicality and economy of grouting during construction.

(5) The lining segments must be designed to withstand transport and construction loads. During storage and transport, segments are usually stacked with strips of timber as separation. Invert segments must withstand uneven loads from muck trains and other loads. The design of invert segments must consider that the segments may not be perfectly bedded. Lining rings used as reaction for shield propulsion must be able to withstand the distributed loads from the jacks, including eccentricities resulting from mismatching adjacent rings.

(6) Joint details must be reinforced to resist chipping and spalling due to erection impact and the effect of uneven jacking on imprecisely placed segments. Tongue-and-groove joints are particularly susceptible to spalling, and the edges of the groove may require reinforcement.

(7) Permanence of the finished structure requires consideration of long-term corrosion and abrasion effects. For a one-pass segmental lining, a high-strength concrete with a high pozzolan replacement is usually desirable for strength, density, tightness, and durability. Precast concrete of 41.4 MPa (6,000-psi) (28-day cylinder) strength or more is routinely used for this purpose. Reinforcement should be as simple as possible, preferably using prefabricated wire mesh.

(8) Once construction and long-term performance requirements have been met, postulated or actual exterior ground or water loads are usually of minor consequence. In rare instances, squeezing ground conditions at great depth may require a thicker lining or higher concrete strength. Water pressures may be reduced by deliberately permitting seepage into the tunnel, and moments in the lining are reduced by using unbolted joints.

5-5. Sequential Excavation and Support

Recognizing the inherent variability of geologic conditions, several construction methods have been developed so that methods of excavation and support can be varied to suit encountered conditions. The most famous of these methods is the New Austrian Tunneling Method (NATM), developed and commonly used in Central Europe. Much older, and applied throughout the world, is the observational method. Both of these methods are discussed in the following sections.

a. NATM.

(1) The so-called NATM is employed for large, noncircular tunnels in poor ground where ground support must be applied rapidly. NATM usually involves the following components:

- Heading-and-bench or multidrift excavation (no shield or TBM).

  Excavation by blasting or, more commonly, by roadheader or other mechanical means.

- Initial ground support usually consisting of a combination of shotcrete, dowels, steel sets, or (now more commonly) lattice girders, installed quickly after exposure by excavation.

- Forepoling or spiling where the ground requires it.

- Stabilizing the face temporarily, using shotcrete and possibly glass-fiber dowels.

- Ground improvement (grouting, freezing, dewatering).

  Extensive use of monitoring to ascertain the stability and rate of convergence of the opening.

(2) The final lining usually consists of reinforced, cast-in-place concrete, often with a waterproofing membrane between the cast-in-place concrete and the initial ground support.

(3) It would appear that the NATM employs virtually all of the means and methods available for tunneling through poor ground. What distinguishes the method is the extensive use of instrumentation and monitoring as an essential part of the construction method. Traditionally, monitoring involves the use of the following devices (see Chapter 9 for additional information about instrumentation and monitoring):

- Convergence measurements, wall to wall and wall to crown.

- Surveying techniques, floor heave, crown sag.

- Multiposition borehole extensometers.

- Strain gages or load cells in the shotcrete, at the rock-shotcrete interface, or on dowels or steel sets, or lattice girders.

(4) The instrumentation is used to assess the stability and state of deformation of the rock mass and the initial ground support and the buildup of loads in or on support components. In the event that displacements maintain their rate or accelerate, that loads build to greater values than support components can sustain, or if instability is visually observed (cracks, distortion), then additional initial ground support is applied. Final lining is placed only after ground movements have virtually stopped.

(5) Initial ground support intensity (number of dowels, thickness of shotcrete, and spacing of steel sets or lattice girders) is applied according to conditions observed and supplemented as determined based on monitoring data. The overall cross section can also be varied according to conditions, changing from straight to curved side walls. The invert can be overexcavated to install a straight or downward curved strut when large lateral forces occur. In addition, sequences of excavation can be changed, for example from heading-and-bench excavation to multiple drifting.

(6) The NATM has been used successfully for the construction of large tunnel cross sections in very poor ground. On a number of occasions, the method has been used even for soft-ground tunnel construction, sometimes supplemented with compressed air in the tunnel for groundwater control and to improve the stand-up time of the ground. Using the NATM in poor rock requires careful execution by contractor personnel well experienced in this type of work. In spite of careful execution, the NATM is not immune to failure. A number of failures, mostly at or near the tunnel face, have been recorded. These have occurred mostly under shallow cover with unexpected geologic or groundwater conditions or due to faulty application insufficient shotcrete strength or thickness, belated placement of ground support, or advancing the excavation before the shotcrete has achieved adequate strength.

(7) It is common to model the complete sequence of excavation and construction using a finite element or finite differences model so as to ascertain that adequate safety factors are obtained for stresses in the final lining. Elastic or inelastic representations of the rock mass properties are used, and tension cracks in unreinforced concrete or shotcrete that propagate to the middle of the cross section are acceptable.

(8) The NATM method of construction requires a special contract format to permit payment for work actually required and carried out and a special working relationship between the contractor and the owner's representative onsite to agree on the ground support required and paid for. Writing detailed and accurate specifications for this type of work is difficult.

(9) While commonly used in Central Europe, the NATM has not been popular in the United States for a number of reasons:

(a) Ground conditions are, for the most part, better in the United States than in those areas of Europe where NATM is popular. In recent years, there have been few opportunities to employ the NATM in the United States.

(b) Typical contracting practices in the United States make this method difficult to administer.

(c) Emphasis in the United States has been on high-speed, highly mechanized tunneling, using conservative ground support design that is relatively insensitive to geologic variations. NATM is not a high-speed tunneling method.

(d) Most contractors and owners in the United States are not experienced in the use of NATM.

This is not meant to imply that the method should not be considered for use in the United States. Short tunnels or chambers (example: underground subway station) located in poor ground that requires rapid support may well be suited for this method. More often, however, the instrumentation and monitoring component of the NATM is dispensed with or relegated to a minor part of the construction method, perhaps applicable only to limited areas of known difficulty. This type of construction is more properly termed "sequential excavation and support.

b. The observational method and sequential excavation and support.

(1) Sequential excavation and support can incorporate some or most of the NATM components, but instrumentation and monitoring are omitted or play a minor role. Instead, a uniform, safe, and rapid excavation and support procedure is adopted for the project for the full length of the tunnel. Or several excavation and support schemes are adopted, each applicable to a portion of the tunnel. The typical application employs a version of the observational method, as follows:

(a) Based on geologic and geotechnical data, the tunnel profile is divided into three to five segments of similar rock quality, where similar ground support can be applied.

(b) Excavation and initial ground support schemes are designed for each of the segments. Excavation options may include full-face advance, heading-and-bench, or multiple drifting. The initial support specification should include designation of maximum time or length of exposure permitted before support is installed.

(c) A method is devised to permit classification of the rock conditions as exposed, in accordance with the excavation and ground support schemes worked out. Sometimes a simplified version of the Q-method of rock mass classification is devised.

(d) Each ground support scheme is priced separately in the bid schedule, using lengths of tunnel to which the schemes are estimated to apply.

(e) During construction the ground is classified as specified, and the contractor is paid in accordance with the unit price bid schedule. The final price may vary from the bid, depending on the actual lengths of different ground classes observed.

(2) The term "sequential excavation and support" is usually employed for excavations that may involve multiple drifting and rapid application of initial support. The observational method works well with this type of construction. However, the observational method also works well with tunneling using TBM. Here, the opening is typically circular, and the initial ground support options do not usually include rapid application of shotcrete, which is considered incompatible with most TBMs. The following is an example of the observational method specified for a TBM-driven tunnel.

(3) Based on the NGI Q-classification system, the rock mass for the Boston Effluent Outfall Sewer Tunnel was divided into three classes: Class A for $Q > 4$; Class B for $4 > Q > 0.4$; and Class C for $Q < 0.4$. Considering that there would be little time and opportunity to permit continuing classification of the rock mass according to the Q-system, a simplified description was adopted for field use:

- Class A typical lower bound description: RQD = 30 percent, two joint sets (one of which associated with bedding planes) plus occasional random joints, joints rough or irregular, planar to undulating, unaltered to slightly altered joint walls, medium water inflow.

- Class B typical lower bound description: RQD = 10 percent, three joint sets, joints slickensided and undulating, or rough and irregular but planar, joint surfaces slightly altered with nonsoftening coatings, large inflow of water.

Class C applies to rock poorer than Class B.

(4) With a TBM-driven tunnel, shotcrete was considered inappropriate, particularly since the types of rock expected would not suffer slaking or other deterioration upon exposure. Maximum use was made of rock dowels, wire mesh, and straps in the form of curved channels, as shown on Figure 5-23 to 5-25. Class A rock might in most instances require no support for the temporary condition; nonetheless, initial ground support was specified to add safety and to minimize the effort required for continuous classification of the rock mass.

(5) The contract also provided for having a number of steel sets on hand for use in the event that bolts or dowels are ineffective in a particular reach. Estimates were made for bidding purposes as to the total aggregate length of tunnel for which each rock class was expected, without specifying where.

(6) For the same project, a short length of smaller tunnel was required to be driven by blasting methods. Two classes of rock were introduced here, equivalent to Class A and Classes B + C (very little if any Class C rock was expected here). Ground supports for these rock classes in the blasted tunnel are shown in Figure 5-26.

5-6. Portal Construction

a. Tunnels usually require a minimum of one to two tunnel diameters of cover before tunneling can safely commence. An open excavation is made to start, which when finished will provide the necessary cover to begin tunneling. Rock reinforcement systems are often used to stabilize the rock cut above the tunnel and are usually combined with a prereinforcement system of dowels installed around the tunnel perimeter to facilitate the initial rounds of excavation (Figure 5-27). If a canopy is to be installed outside of the tunnel portal for protection from rock falls, it should be installed soon after the portal excavation has been completed. If multiple stage tunnel excavation is to be used on the project, the contractor may excavate the portal only down to the top heading level and commence tunneling before taking the portal excavation down to the final grade.

b. Tunnel excavation from the portal should be done carefully and judiciously. Controlled blasting techniques should be used and short rounds of about 1 m in depth are adequate to start. After the tunnel has been excavated to two or three diameters from the portal face, or as geology dictates, the blasting rounds can be increased progressively to standard length rounds used for normal tunneling.

c. When constructing portals, the following special issues should be accounted for:

(1) The rock in the portal is likely to be more weathered and fractured than the rock of the main part of the tunnel.

(2) The portal must be designed with proper regard for slope stability considerations, since the portal excavation will unload the toe of the slope.

(3) The portal will be excavated at the beginning of mining before the crew has developed a good working relationship and experience.

(4) The slope must be adequately designed to adjust to unloading and stress relaxation deformations.

(5) The portal will be a heavily used area, and a conservative design approach should be taken because of the potential negative effects instability would have on the tunneling operations.

d. The design of portal reinforcement will depend on geologic conditions. Rock slope stability methods should be used unless the slope is weathered or under a deep layer of overburden soil. In this case, soil slope stability analyses must be performed for the soil materials. Often, both types of materials are present, which will require a combined analysis.

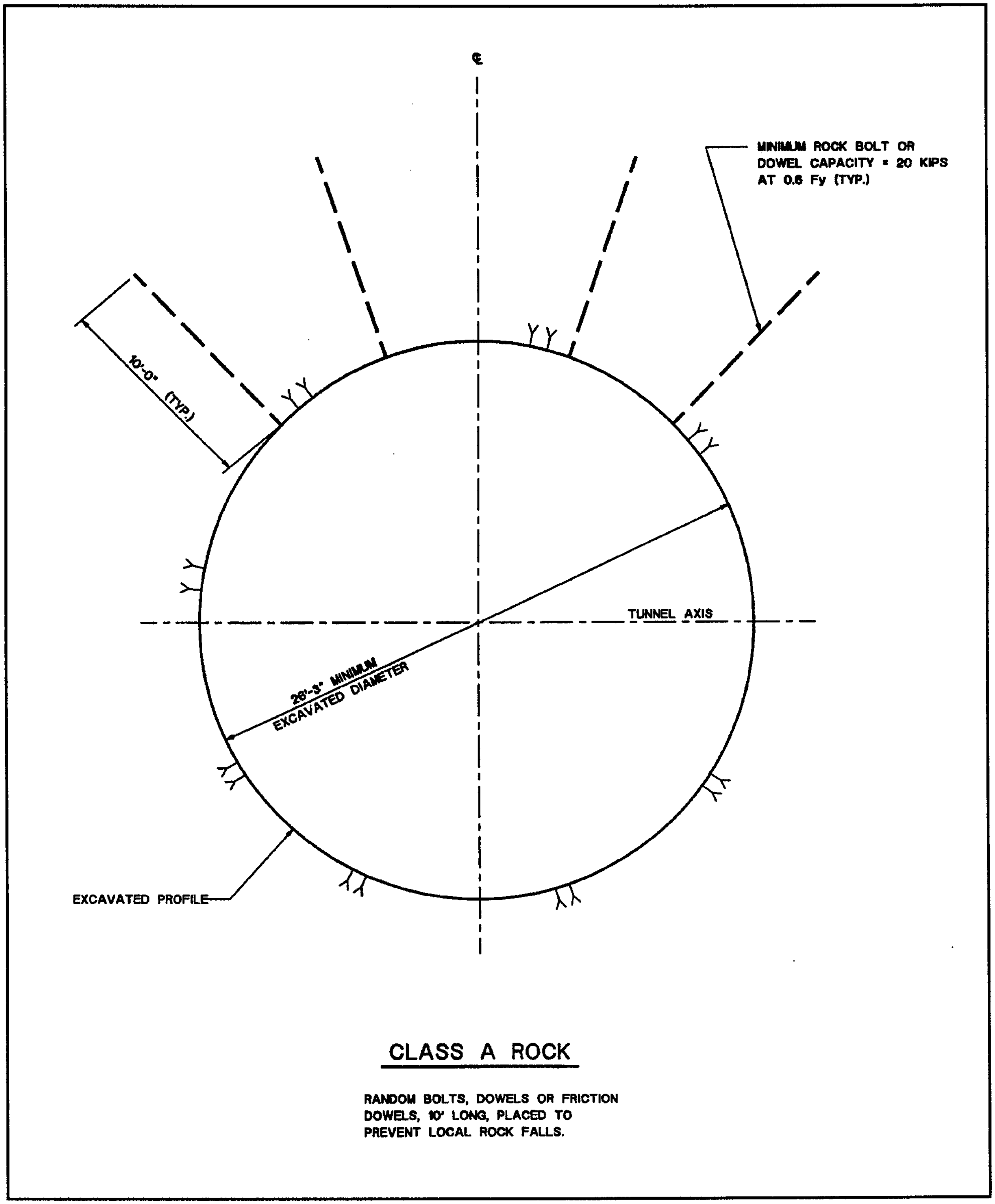


Figure 5-23. Ground support, Class A rock

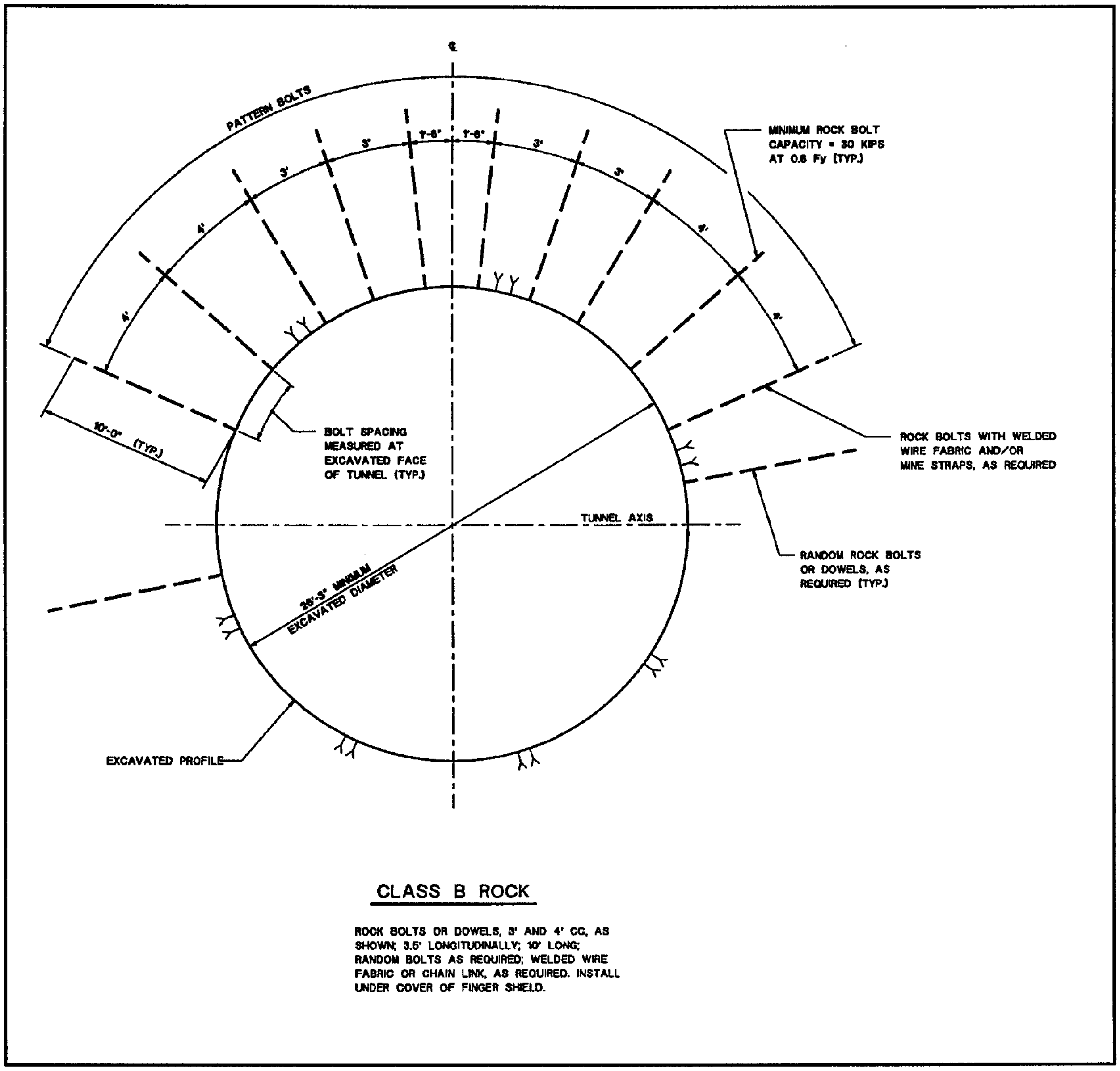


Figure 5-24. Ground support, Class B rock

e. The types of portal treatments that may be considered include the following:

- No support at the portal when excellent geologic conditions prevail.
- Portal canopy only for rock fall protection.
- Rock reinforcement consisting of a combination of rock bolts, steel mesh, shotcrete, and weeps.
- Rock reinforcement and a canopy for very poor conditions.

Tunnel reinforcement is usually more intense in the vicinity of the portal until the effects of the portal excavation are no longer felt.

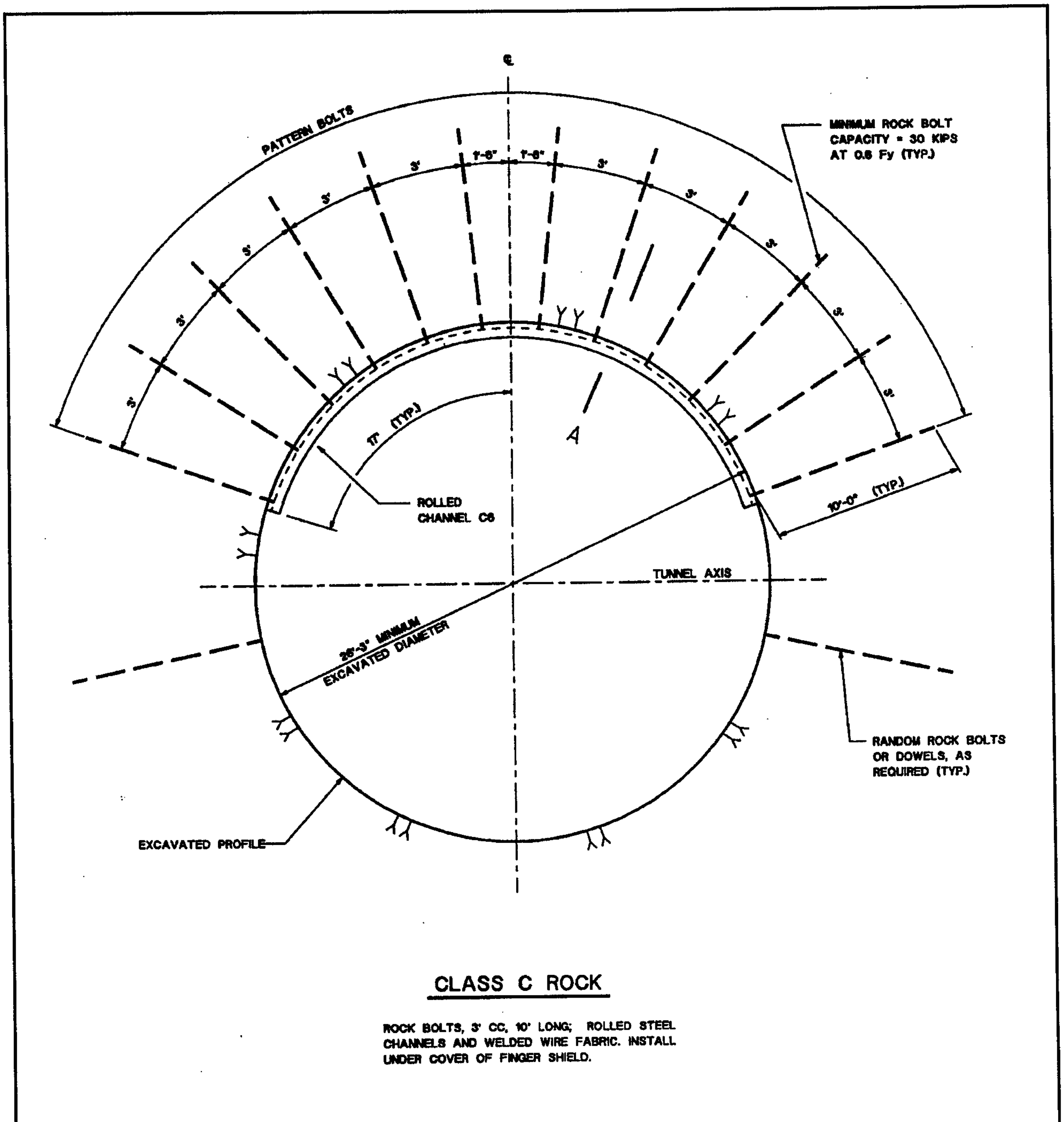


Figure 5-25. Ground support, Class C rock

5-7. Shaft Construction

Most underground works include at least one deep excavation or shaft for temporary access or as part of the permanent facility. Shafts typically go through a variety of ground conditions, beginning with overburden excavation, weathered rock, and unweathered rock of various types, with increasing groundwater pressure. Shaft construction options are so numerous that it is not possible to cover all of them in this manual. The reader is referred to standard foundation engineering texts for shaft construction,

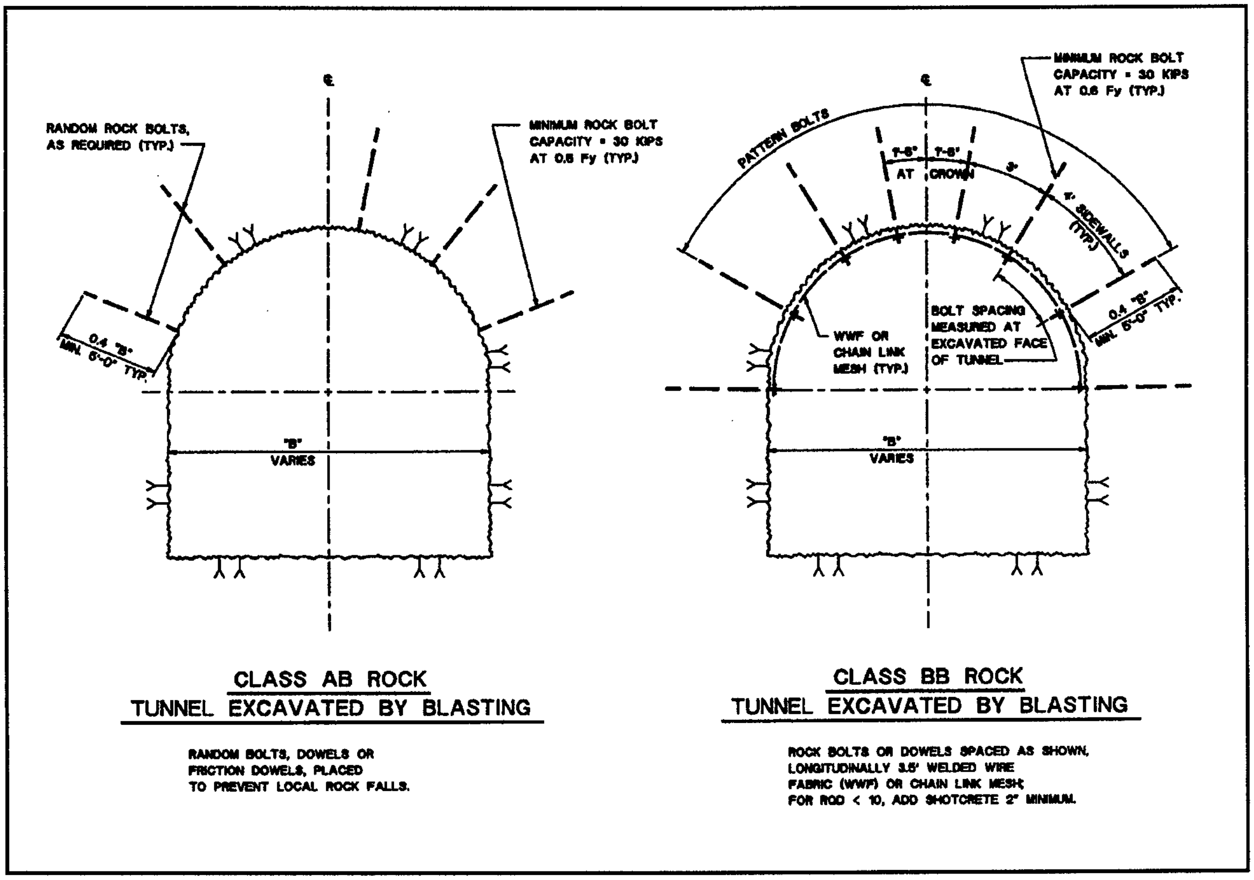


Figure 5-26. Ground support, blasted tunnel

temporary and permanent walls through soil and weathered rock, and to the mining literature for deep shafts through rock. The most common methods of shaft excavation and ground support are summarized in this section.

a. Sizes and shapes of shafts. Shafts serving permanent functions (personnel access, ventilation or utilities, drop shaft, de-airing, surge chamber, etc.) are sized for their ultimate purpose. If the shafts are used for construction purposes, size may depend on the type of equipment that must use the shaft. Shallow shafts through overburden are often large and rectangular in shape. If space is available, a ramp with a 10-percent grade is often cost-effective. Deeper shafts servicing tunnel construction are most often circular in shape with a diameter as small as possible, considering the services required for the tunnel work (hoisting, mucking, utilities, etc.). Typical diameters are between 5 and 10 m (16-33 ft). If a TBM is used, the shaft must be able to accommodate the largest single component of the TBM, usually the main bearing, which is usually of a size about two-thirds of the TBM diameter.

b. Shaft excavation and support through soil overburden.

(1) Large excavations are accomplished using conventional soil excavation methods such as backhoes and dozers, supported by cranes for muck removal. In hard soils and weathered rock, dozers may require rippers to loosen the ground. The excavation size will pose limits to the maneuverability of the excavation equipment.

(2) Smaller shafts in good ground, where groundwater is not a problem, can be excavated using dry drilling methods. Augers and bucket excavators mounted on a kelly, operated by a crane-mounted torque table attachment, can drill holes up to some 75-m (250-ft) depth and 8-m (25-ft) diam. A modified oil derrick, equipped with an elevated substructure and a high-capacity torque table, is also effective for this type of drilling.

(3) Many options are available for initial ground support, including at least the following:

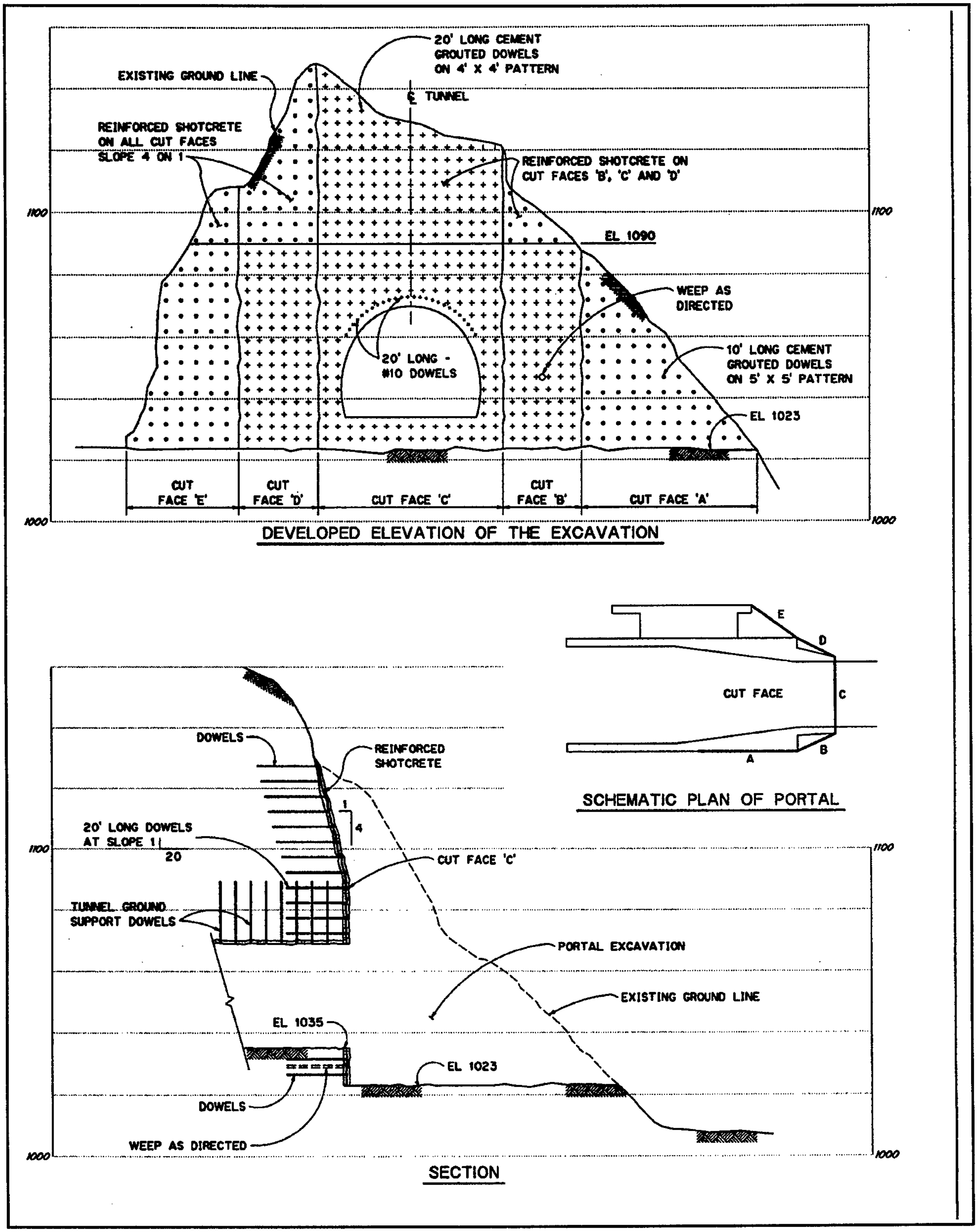


Figure 5-27. Portal excavation and support (H-3 tunnel, Oahu)

- Soldier piles and lagging, in soils where groundwater is not a problem or is controlled by dewatering.

- Ring beams and lagging or liner plate.

- Precast concrete segmental shaft lining.

- Steel sheet pile walls, often used in wet ground that is not too hard for driving the sheet piles.

- Diaphragm walls cast in slurry trenches; generally more expensive but used where they can have a permanent function or where ground settlements and dewatering must be controlled.

- Secant pile walls or soil-mixing walls as substitutes for diaphragm walls, but generally less expensive where they can be used.

(4) Circular shafts made with diaphragm or secant pile walls usually do not require internal bracing or anchor support, provided circularity and continuity of the wall is well controlled. Other walls, whether circular or rectangular, usually require horizontal support, such as ring beams for circular shafts, wales and struts for rectangular shafts, or soil or rock anchors or tiebacks that provide more open space to work conveniently within the shaft.

(5) In good ground above the groundwater table, soil nailing with shotcrete is often a viable ground support alternative.

c. Shaft excavation through rock.

(1) Dry shaft drilling using a crane attachment or a derrick, as briefly described in the previous subsection, has been proven viable also in rock of strength up to 15 MPa (2,200 psi), provided that the ground is initially stable without support. Use of a bucket with extendable reamer arms permits installation of initial ground support, which would consist of shotcrete and dowels as the shaft is deepened.

(2) Deep shafts can be drilled using wet, reverse circulation drilling. Drilling mud is used to maintain stability of the borehole and counterbalance the formation water pressure, as well as to remove drill cuttings. The drilling is done with a cutterhead, furnished with carbide button cutters and weighted with large donut weights to provide a load on the cutterhead. The drill string is kept in tension, so that the pendulum effect can assist in maintaining verticality of the borehole. Mud is circulated by injecting compressed air inside the drill string; this reduces the density of the drilling mud inside the string and forces mud and drill cuttings up the string, through a swivel, and into a mud pond. From there the mud is reconditioned and led back into the borehole. This type of shaft construction usually requires the installation of a steel lining or casing with external stiffeners, grouted in place. If the steel casing is too heavy to be lowered with the available hoisting gear, it is often floated in with a bottom closure and filled partly with water. This method permits shafts of 2-m (7-ft) diam to be constructed to depths of about 1,000 m (3,300 ft). Larger diameters can be achieved at shallower depths.

(3) If underground access is available, shafts can be drilled using the raise drilling method. A pilot bore is drilled down to the existing underground opening. Then a drill string is lowered, and a drillhead is attached from below. The string is turned under tension using a raise drill at the ground surface, and the shaft is created by backreaming, while cuttings drop into the shaft to the bottom, where they are removed. This method requires stable ground. Raise boring can also be used for nonvertical shafts or inclines. A raised bore can be enlarged using the slashing method of blasting. The bore acts as a large burn cut, permitting blasting with great efficiency and low powder factors.

(4) Conventional shaft sinking using blasting techniques can be used to construct a shaft of virtually any depth, size, and shape. A circular shape is usually preferred, because the circular shape is most favorable for opening stability and lining design. Shaft blasting tends to be more difficult and more confined than tunnel blasting. Typically, shorter rounds are pulled, and the powder factor is greater than for a tunnel in the same material. Variations of the wedge cut are used rather than the burn cut typically used for tunnels. Shallow shaft construction can be serviced with cranes, but deeper shaft construction requires more elaborate equipment. The typical arrangement includes a headframe at the top suspending a two- or three-story stage with working platforms for drilling and blasting, equipment for mucking, initial ground support installation, and shaft lining placement. The typical shaft lining is a cast-in-place concrete lining, placed 10 to 15 m (33-50 ft) above the advancing face.

(5) If the shaft is large enough to accommodate a roadheader, and the rock is not too hard, shaft excavation can be accomplished without explosives using crane service or headframe and stage equipment.

(6) Most shaft construction requires the initial construction of a shaft collar structure that supports overburden and weathered rock near the surface and construction loads adjacent to the top of the shaft. It also serves as a foundation for the temporary headframe used for construction as well as for permanent installations at the top of the shaft.

(7) Inclines of slopes up to about 25 deg can be bored using a TBM specially equipped to maintain its position in the sloping hole. Inclines at any angle can be excavated using blasting methods, with the help of climbing gear such as the Alimak climber.

5-8. Options for Ground Improvement

When difficult tunnel or shaft construction conditions are foreseen, ground improvements are often advisable and sometimes necessary. There are, generally speaking, three types of ground improvement that can be feasibly employed for underground works in rock formations:

- Dewatering.
- Grouting.
- Freezing.

a. Ground improvement for shaft sinking.

(1) Ground improvement must be considered when shaft sinking involves unstable ground associated with significant groundwater inflow. At a shallow depth, groundwater is often found in potentially unstable, granular materials, frequently just above the top of rock. If sufficiently shallow, the best solution is to extend the shaft collar, consisting of a nominally tight wall, into the top of rock. Shallow groundwater can also often be controlled by dewatering.

(2) An exploratory borehole should be drilled at or close to the center of all shafts. Borehole permeability (packer) tests can be used to estimate the potential groundwater inflow during construction that could occur if the groundwater were not controlled. If the estimated inflow is excessive, ground improvement is called for. At the same time, core samples will give an indication of ground stability as affected by groundwater inflow. Poorly cemented granular sediments and shatter zones are signs of potential instability.

(3) Deep groundwater usually cannot be controlled by dewatering; however, grouting or freezing can be tried. This is usually done from the ground surface before shaft sinking commences, because it is very costly to work down the shaft. Both methods require the drilling of boreholes for the installation of freeze pipes or for grouting. When the shaft is very deep, high-precision drilling is required to reduce the deviation of boreholes to acceptable magnitudes. Considering that borehole spacings are of the order of 1.5 to 2 m (6-7 ft) and that both grouting and freezing rely on accurate placement of the holes, it is readily appreciated that even a deviation of 1 m can be critical. Nonetheless, freezing and grouting have been successfully carried out to depths greater than 500 m (1,700 ft). It is also readily appreciated that both grouting and freezing are very costly; however, they are often the only solutions to a serious potential problem.

(4) Freezing is often more expensive than grouting, and it takes some time to establish a reliable freeze wall, while grouting can be performed more quickly. Professionals in the shaft sinking business generally consider freezing to be substantially more reliable and effective than grouting. It is not possible to obtain a perfect grout job—a substantial reduction of permeability (say, 80-90 percent) is the best that can be hoped for—and grouting may leave some areas ungrouted. On the other hand, a freeze job can more readily be verified and is more likely to create a continuous frozen structure, thus is potentially more reliable.

(a) *Grouting*. General advice and design recommendations for grouting are found, for example, in EM 1110-2-3506, Grouting Technology, and in Association Française (1991). The detailed grouting design for deep shafts is often left to a specialist contractor to perform and implement. While chemical grouting is often used in loose sediments and overburden materials, grouting in rock is usually with cement. Grout penetration into fractures is limited by aperture of the fractures relative to the cement particle sizes. As a rule, if the rock formation is too tight to grout, it is also usually tight enough that groundwater flow is not a problem. Shaft grouting typically starts with the drilling of two or three rows of grout holes around the shaft perimeter, spaced 1.5 to 2.0 m (5-7 ft) apart. Grout injection is performed in the required zones usually from the bottom up, using packers. The effectiveness of the grout job can be verified by judicious sequencing of drilling and grouting. If secondary grout holes drilled after the first series of grouted holes display little or no grout take, this is a sign of the effectiveness of grouting. Additional grout holes can be drilled and grouted as required, until results are satisfactory. If it becomes necessary to grout from the bottom of the shaft, indicated, for example, by probeholes drilled ahead of the advancing shaft, then grout

holes are drilled in a fan pattern covering the stratum to be grouted. It is important to perform the grouting before a condition has arisen with large inflows, because grouting of fissures with rapidly flowing water is very difficult. When drilling from the bottom of a deep shaft, it is often necessary to drill through packers or stuffing boxes to prevent high-pressure water from entering the shaft through the drillholes.

(b) *Freezing.* Brine is usually used as the agent to withdraw caloric energy from the ground and freeze the water in the ground. The brine is circulated from the refrigeration plant in tubes placed in holes drilled through the ground to be frozen. The tubes can be insulated through ground that is not intended to be frozen. The detailed design and execution of a freezing program requires specialist knowledge and experience that is only available from firms that specialize in this type of work. The designer of the underground work should prepare a performance specification and leave the rest to the contractor and his specialist subcontractor. The detailed design of a freeze job includes the complete layout of plant and all freeze pipes so as to achieve a freeze wall of adequate strength and thickness and thermal analyses to estimate the required energy consumption and the time required to achieve the required results, with appropriate safety factors. The English-language literature does not offer a great number of references on ground freezing. One source is the Proceedings of the Third International Symposium on Ground Freezing (USACE 1982). The strength of frozen ground is dependent on the character and water content of the ground and increases with decreasing temperature of the frozen ground. Some rock types, notably weak, fine-grained rocks, suffer a substantial strength loss upon thawing. The effects of thawing must be considered in the design of the final shaft lining. Saline groundwater is more difficult to freeze because of its lower freezing temperature. If the formation water is not stagnant but moves at an appreciable rate, it will supply new caloric energy and delay the completion of the freeze job. The velocity of formation water movement should be estimated ahead of time, based on available head and gradient data. At the ground surface, brine distribution pipes are often laid in a covered trench or gallery around the shaft, keeping them out of the way from shaft construction activities. Since freezing involves expansion of the formation water, a relief borehole is usually provided at the center of the shaft so that displaced water can escape. The freezing process is controlled by installing temperature gages at appropriate locations between freeze pipes, as well as through monitoring of the temperature of return brine and the overall energy consumption. On rare occasions it becomes necessary to implement a freezing installation from the bottom of the shaft. This usually requires the construction of a freezing gallery encircling the shaft. Shaft excavation cannot proceed during the implementation of an underground freeze job, including the time required to achieve the necessary reduction in ground temperature. Down-the-shaft freezing, therefore, is very costly. Quicker implementation of a freezing application can be accomplished using liquid nitrogen as coolant rather than brine.

b. Ground improvement for tunneling. Rock tunnels generally do not require ground improvement as frequently as shafts. Examples of ground improvements using grout applications are briefly described in the following.

(1) *Preconstruction application.* Where it is known that the tunnel will traverse weak ground, such as unconsolidated or poorly consolidated ground or a wide shatter zone, with high water pressure, the ground can be grouted ahead of time. It is preferable to grout from the ground surface, if possible, to avoid delaying tunneling operations. Such grout applications are particularly helpful if the water is contaminated with pollutants or if the groundwater is hot. The primary purpose of applying grout is to reduce the ground's permeability. Strengthening of the ground is sometimes a side benefit.

(2) *Application during construction.* When grouting cannot be applied from the ground surface, it can be carried out from the face of the tunnel before the tunnel reaches the region with the adverse condition. An arrangement of grout holes are drilled in fan shape some 20 to 40 m (60-130 ft) ahead of the face. Quality control is achieved by drilling probeholes and testing the reduction of permeability. Grouting is continued until a satisfactory permeability reduction is achieved.

(3) *Application after probehole drilling.* Where adverse conditions are expected but their location is unknown, probehole drilling will help determine their location and characteristics. Such probeholes can be simple percussion holes with a record of water inflow, or packer tests can be performed in these probeholes. The grout application can be designed based on the results of one or more probeholes.

(4) *Postexcavation grouting.* If it is found that water inflow into the excavated tunnel is too large for convenient placement of the final lining, radial grouting can be performed to reduce the inflow. Generally, the grout is first injected some distance from the tunnel, where water flow velocities are likely to be smaller than at closer distances. It is sometimes necessary to perform radial grouting after the completion of the tunnel lining. Here, the finished

lining helps to confine the grout, but the lining must be designed to resist the grout pressures.

(5) *Freezing in tunnels.* Freezing is sometimes a suitable alternative to grouting for temporary ground strengthening and inflow control. Freezing is particularly effective if the ground is weak, yet too impervious for effective grout penetration.

5-9. Drainage and Control of Groundwater

a. General. The design of a permanent drainage system and the control systems required for groundwater begins during the geotechnical exploration phases with an assessment of the potential sources and volumes of water expected during construction. The type of permanent drainage system required will depend upon the type of tunnel and site groundwater conditions.

b. Assessment of water control requirements. Prior to construction, estimates of the expected sources of groundwater and the expected inflow rates and volumes must be identified in order for the contractor to provide adequate facility for handling inflow volumes. Section 3-5 provides guidance in identifying potential sources of groundwater and for making inflow volume estimates.

c. Care of groundwater during construction.

(1) Care of groundwater generally is the responsibility of the contractor; however, the specifications for a tunnel contract may require that certain procedures be followed. For example, if it is expected that water-bearing joints will be present that contain sufficient head and volume to endanger the safety of the tunnel, the drilling of a probe-hole ahead of the working face should be required. The following discussion is for guidance.

(2) Water occurring in a tunnel during construction must be disposed of because it is a nuisance to workers and may make the placement of linings difficult or cause early weakening of the linings. It also makes the rock more susceptible to fallout by reducing the natural cohesion of fine-grained constituents.

(3) The excavation sequence should be such that drainage of the sections to be excavated is accomplished before excavation. Thus, a pilot drift near the invert in a wet environment is more effective than a top heading although enlargement to full size is more difficult. It is an excellent practice to carry a drill hole three tunnel diameters in advance of the working face. The drill hole has an additional advantage of revealing rock conditions more clearly than defined by the initial investigation.

(4) When encountered, water should be channeled to minimize its effect on the remaining work. To accomplish this, the surface of a fissure may be packed with quick-setting mortar around a tube leading to a channel in the invert. Ingenuity on the part of workers and supervisors can produce quick, effective action and should be encouraged so long as objectionable materials do not intrude within the concrete design line.

(5) If groundwater inflow is extremely heavy and drainage cannot be accomplished effectively, it will be necessary to install a "grout umbrella" from the face before each tunnel advance is made. This consists of a series of holes angled forward and outward around the perimeter of the face that are pumped with grout to fill fractures and form a tunnel barrier against high inflows.

(6) For permanent protection from the flow of water along the outside of the concrete lining, no better method exists than filling with grout any void that remains after the concrete is set.

(7) Section 5-14.b. provides additional information on the control and disposal of groundwater.

d. Permanent drainage systems.

(1) *Drainage system.* The drainage system required in a tunnel will depend on the type of tunnel, its depth, and groundwater conditions. Some tunnels may not require special drainage. Others may require drainage to limit the pressure behind the lining or to remove water due to condensation and leakage through the tunnel joints. A detailed design procedure for drains will not be attempted here; however, a brief description will be included to indicate what is involved in providing drainage for the various types of tunnels.

(a) *Pressure tunnels.* Drainage for pressure tunnels may be required if normal outlets through gates or power units do not accomplish complete unwatering of the tunnel. The drains are then located at the low point of the tunnel and are provided with a shutoff valve. In some cases, it is desirable to provide drainage around a pressure tunnel. This may be done to limit the external head on the lining or to limit pressures in a slope in the event leakage developed through the lining. Drainage may be provided by drilling holes from the downstream portal or by a separate drainage tunnel.

(b) *Outlet tunnels.* Drainage for outlet tunnels may be required to completely unwater the tunnel if some point along the tunnel is lower than the outlet end. To limit the external head, drains can be provided that lead directly into the tunnel. In this manner, the outlet tunnel also serves as a drain tunnel.

(c) *Vehicular tunnels.* Drainage for vehicular tunnels will usually consist of weep holes to limit the pressure behind the lining and an interior drain system to collect water from condensation and leakage through the joints in the lining. Interior drainage can be either located in the center of the tunnel between vehicular wheel tracks or along the curbs. If the tunnel is located in areas where freezing temperatures occur during part of the year, precautions should be taken to prevent freezing of the drains. If the tunnel is long, protection against freezing need not be installed along the entire length of tunnel, depending on the climate and depth at which the tunnel is located.

(d) *Drain and access tunnels.* Drainage from these tunnels may require a sump and pump, depending on the location of the outlet end. Drain tunnels usually have drain holes that extend from the tunnel through the strata to be drained.

(e) *Waterstop.* To prevent uncontrolled water seepage into a concrete-lined tunnel, the construction joints are waterstopped. EM 1110-2-2102 covers the types and use of waterstops.

(2) *Grouting.* Grouting in connection with tunnel construction is covered in paragraph 28 and Plate 5 of EM 1110-2-3506. Recommendations are made below regarding special grouting treatment typically required to prevent drainage problems in various types of tunnels or shafts. Ring grouting (i.e., grouting through radial holes drilled into the rock at intervals around the tunnel periphery) is used to reduce the possibility of water percolating from the reservoir along the tunnel bore and for consolidation grouting along pressure tunnels. Contact grouting refers to the filling of voids between concrete and rock surface with grout.

(a) *Outlet works tunnels.* As a minimum, the crown of outlet works tunnels should be contact grouted for their entire length. Grouting to prevent water from percolating along the tunnel bore should consist of a minimum of one ring, interlocked with the embankment grout curtain. If the impervious core of the embankment extends upstream from the grout curtain and sufficient impervious material is available between the tunnel and the base of the embankment, the location near the upstream edge of the impervious core also should extend into the rock approximately one tunnel diameter.

(b) *Pressure tunnels.* Pressure tunnel linings are designed in two ways. Either the concrete and steel linings act together to resist the entire internal pressure or concrete and steel linings and the surrounding rock act together to resist the internal pressure. Contact and ring grouting for pressure tunnels is done the same as for outlet tunnels except one additional ring should be grouted at the upstream end of the steel liner. Consolidation grouting of the rock around the lining of a pressure tunnel and the filling of all voids is a necessity if the rock is to take part of the radial load. Consolidation grouting of the rock behind the steel liner is good practice and should be done whether or not the rock is assumed to resist a portion of the internal pressure.

(c) *Shafts.* Shafts are normally grouted the same as tunnels except that grouting is done completely around the shaft in all cases.

5-10. Construction of Final, Permanent Tunnel Linings

When the initial ground support components described in the previous sections do not fulfill the long-term functional requirements for the tunnel, a final lining is installed. On occasion, an initial ground support consisting of precast segments will also serve as the final lining (see Section 5-4.i). More typically, the final lining will be constructed of cast-in-place concrete, reinforced or unreinforced, or a steel lining surrounded by concrete or grout. Guidelines for the selection of a final lining is presented in Section 9-1. The following subsections describe cast-in-place concrete lining and steel lining construction.

a. Cast-in-place concrete lining. When a concrete lining is required, the type most commonly used is the cast-in-place lining. This lining provides a hydraulically smooth inside surface, is relatively watertight, and is usually cost competitive. Concrete linings can be of the following types:

- Unreinforced concrete.
- Concrete reinforced with one layer of steel, largely for crack control.
- Concrete reinforced with two layers of steel, for crack control and bending stresses.

- Unreinforced or reinforced concrete over full waterproofing membrane.

(1) *Placement sequence.* Depending on tunnel size and other factors, the entire cross section is placed at one time, or the invert is placed first, or the invert is placed last. Sometimes precast segments are placed in the invert to protect a sensitive rock from the effects of tunnel traffic, followed by placement of the crown concrete. This method will leave joints between the invert segments, but these joints can be designed for sealing or caulking. Barring construction logistics constraints, the most efficient method of placement is the full-circle concreting operation. When schedule or other constraints require that concrete be placed simultaneously with tunnel excavation and muck removal through the tunnel segment being concreted, then either the precast-invert segment method or the arch-first method is appropriate. Depending on the tunnel size, the upper 270 deg of a circular tunnel are placed first to permit construction traffic to flow uninterrupted and concurrently with lining placement. With the precast-invert segment method, the segment is made wide enough to permit all traffic operations. The invert-first placement method is not now frequently used for circular tunnels, in part because the invert takes time to cure and is subject to damage during placement of the crown. This method is sometimes advantageous when a waterproofing membrane is used. When the final lining is horseshoe-shaped, the invert is usually placed first, furnished with curbs to guide the placement of sidewall forms. Sometimes, especially in tunnels with ribs as initial ground support, L-shaped wall foundations are placed first; these will then guide the placement of the invert and the side walls.

(2) *Formwork.* Except for special shapes at turns and intersections, steel forms are used exclusively for tunnels of all sizes. The forms often come in widths of 1.5 to 1.8 m (5-6 ft), with provisions to add curve filler pieces to accommodate alignment radii. The segments are hinged and collapsible to permit stripping, transporting, and reerection, using special form carriers that ride on rails or rubber tires. The forms are usually equipped with external vibrators along with provisions to use internal vibrators through the inspection ports if necessary. Telescoping forms permit leapfrogging of forms for virtually continuous concrete placement.

(3) *Concrete placement.* Placement is accomplished using either of two methods: the conventional slick line method and the multiport injection method.

(a) The slick line is a concrete placement pipe, 150 to 200 mm (6-8 in.) in size, placed in the crown from the open end of the form up to the previously placed concrete. Concrete is pumped into the form space until a sloping face of the fresh concrete is created in the form space. The slick line is gradually withdrawn, keeping the end of the pipe within the advancing fresh concrete. Minimum depth of pipe burial varies between 1 and 3 m (3-10 ft), depending on size of tunnel and thickness of lining. The advancement of the concrete is monitored through inspection ports and vibrated using form vibrators and internal vibrators.

(b) With the injection method, special injection ports are built into the form, through which concrete is placed using portable pumping equipment. Again, placement occurs in the direction from the previously placed concrete. Depending on the diameter of the tunnel, one to five injection ports may be located at any given cross section, with one port always at the crown. For large-diameter tunnels, and for reinforced linings, it is inadvisable to let the fresh concrete fall from the crown to the invert. Here, concrete must be placed through ports. Concrete forms are usually stripped within 12 hr of placement so as to permit placement of a full form length every day. Concrete must have achieved enough strength at this time to be self-supporting. Usually a strength of about 8.3 MPa (1,200 psi) is sufficient.

(4) *Groundwater control during concreting.* Water seepage into the tunnel may damage fresh concrete before it sets. Side wall flow guides, piping, and invert drains may be used to control water temporarily. After completion of the lining, such drain facilities should be grouted tight. High-water flows may require damming or pumping, or both, to remove water before placing concrete. On occasion, formation grouting may be required.

(5) *Concrete conveyance.* The concrete is brought from the surface to the tunnel level either by pumping or through a drop pipe. If conveyed through a drop pipe, the concrete is remixed to eliminate separation. If the concrete is pumped, the pumping may continue through the tunnel all the way to the point of placement. Depending on the distance, booster pumps may be used. If possible, additional shafts are placed along the tunnel to reduce the distance of concrete conveyance in the tunnel. Other conveyance methods in the tunnel include conveyors, agitator cars, or nonagitated cars, trammed by locomotives to the point of placement. Remixing may be required, depending on the system used, to maintain the proper consistency of the fresh concrete. It is also possible at this location to add an accelerator if necessary. When conveying concrete for long distances, it is possible to add a retarder to

maintain fluidity, then supplemented with an accelerator prior to placement.

(6) *Construction joints*. Transverse joints are located between pours, often 30 m (100 ft) apart or up to nearly 60 m (200 ft), depending on the form length used by the contractor. Either a sloping joint or a vertical joint can be used. Either type will result in a structurally acceptable joint. When a sloping joint is used, a low bulkhead is usually used to limit the feathering out of the concrete at the invert. The advantage of the sloping joint is that only a low bulkhead is required; this method is least likely to result in voids when using a slick line method. Disadvantages of the sloping joint include the following:

- Difficulty in proper preparation of joints before the next pour.
- Waterstop placement not feasible.
- Underutilization of total length of the form.
- Formation of much longer construction joint, compared with the vertical joint.

The sloping joint is often more convenient when an unreinforced lining is constructed. The advantages of the vertical joint are accessibility of the joints for proper preparation, formation of the shortest possible length of joint, and full utilization of formwork. The vertical joint is most often used with reinforced concrete linings. Some of the disadvantages include the additional time required for bulkhead installation, provisions for maintaining reinforcing steel continuity across the joint, and the probability of forming voids when using the slick line method. From the perspective of watertightness, longitudinal joints resulting from the two-pour methods are not desirable. In particular, the arch-first method poses the greatest difficulty in joint surface treatment to achieve desired watertightness. Waterstops are not used for construction joints in unreinforced concrete linings. Water stops and expansion joints are of doubtful value in reinforced concrete linings but are sometimes used at special locations, such as at changes in shape of opening, intersections, and transitions to steel-lined tunnels.

(7) *Contact grouting*. When a tunnel lining has to withstand appreciable loads, external or internal, it is essential that the lining acts uniformly with the surrounding rock mass, providing uniformity of loading and ground reaction. Hence, significant voids cannot be tolerated. Voids are often the result of imperfect concrete placement in the crown. Voids are virtually unavoidable in blasted tunnels with irregular overbreak. It is therefore standard practice to perform contact grouting in the crown, using groutholes that have been either preplaced or drilled through the finished lining, so as to fill any crown voids that remain. Grouting is usually made to cover the upper 120 to 180 deg of circumference, depending on tunnel size and amount of overbreak. USACE has a guide specification for Tunnel and Shaft Grouting, available from HQUSACE.

(8) *Supplementary grouting and repair*. In the event that groundwater leaks excessively into the finished tunnel, formation grouting can be used to tighten the ground. This is done through radial groutholes through the lining. Leaking joints can also be repaired by grouting or epoxy treatment.

b. *Steel lining*. A steel lining is required when leakage through a cracked concrete lining can result in hydrofracturing of the surrounding rock mass or deleterious leakage or water loss. In most respects, the steel lining is similar to open-air penstocks, except that the tunnel steel lining is usually designed for an exterior water pressure and is furnished with external stiffeners for high external pressure conditions. Fabrication and assembly of a steel lining generally follow the same standards and practices as penstocks described in American Society of Civil Engineers (ASCE) (1993). Some construction aspects of steel-lined tunnels, however, deserve special attention, particularly as they affect the preparation of contract documents.

(1) *Constructibility*. Individual pipes and joints are usually made as large as can be practically transported on the highway to the site and into the tunnel for placement and joining, leaving field welding to a minimum. Each motion through shafts, adits, and tunnel must be considered in the evaluation of the maximum size of the individual pieces.

(2) *Handling and support*. Pipes without external stiffeners should be internally supported during transport and installation if their diameter/thickness ratio, D/t, is less than 120. The internal bracing can be timber or steel stulling (see ASCE 1993) or spiders with adjustable rods. The minimum thickness of the steel shell is usually taken as $t_{min} = (D + 20)/400$, with dimensions in inches, or more simply $t_{min} = D/350$ (in inches or millimeters). Externally coated pipes must be protected from damage to coating, using appropriate support and handling, e.g., fabric slings.

(3) *Support during concrete placement.* The pipe must be centrally aligned in the excavated tunnel and prevented from distortion and motion during concrete placement. This may require the pipe to be placed on cradles, usually of concrete, with tiedowns to hold the pipe in place against flotation and internal stulling. Steel or concrete blocking (not timber) is often used to resist flotation.

(4) *Jointing.* Welding procedures, including testing of welds, are similar to those of surface penstocks. It is often impractical to access the exterior of the pipe for welding and testing. An external backup ring, though less satisfactory, may be required. All welds should be tested using nondestructive testing methods using standards of acceptance similar to surface penstocks (see ASCE 1993).

(5) *Concrete placement.* The tunnel must be properly prepared for concrete placement. Because the concrete must provide a firm contact between steel and ground, all loose rock and deleterious materials, including wood blocking, must be removed and groundwater inflow controlled as discussed in the previous subsection. Adequate clearances must be provided around the pipe. The concrete is usually placed using the slick line method. The concrete mix should be selected to minimize the buildup of heat due to hydration; subsequent cooling will result in the creation of a thin void around the pipe. Usually a relatively low strength (14 MPa, 2,000 psi, at 28 days) is adequate. Sloping cold joints are usually permissible.

(6) *Contact grouting.* Grouting applications include the filling of all voids between concrete backfill and rock, which is termed contact grouting, and skin grouting of the thin void between steel lining and concrete. Contact grouting is often carried out through grout plugs provided in the pipe, located at the top and down 15 and 60 deg on each side to cover the upper 180 deg of installation. The grout plugs are spaced longitudinally every 3 m (10 ft), staggered, or between stiffeners if the pipe has external stiffeners. Grout holes are drilled through the predrilled holes in the steel plate, the concrete, and up to about 600 mm (2 ft) into the surrounding rock. The grout is a sand-cement mix, applied at pressures up to 0.7 MPa (100 psi).

(7) *Skin grouting.* The purpose of skin grouting is to fill the thin void that may exist between concrete and steel after the concrete cures. Theoretically, skin grouting is not required if a conservative value of the void thickness has been assumed in design, and a safe and economical structure can be achieved without skin grouting. If skin grouting is to be performed, it is usually according to the following procedure:

(a) After curing of the concrete (days or weeks), sound the steel for apparent voids and mark the voids on the steel surface.

(b) Drill 12- to 18-mm (0.5- to 0.75-in.) holes at the lower and the upper part of the voids.

(c) Grout with a flowable nonshrink grout, using the upper hole as a vent.

(d) After grout has set, plug holes with threaded plugs and cap with a welded stainless steel plate.

5-11. Ventilation of Tunnels and Shafts

Shaft and tunnel construction generally occurs in closed, dead-end spaces, and forced ventilation is essential to the safety of the works. Specifically, the Occupational Health and Safety Act (OSHA) 10 CFR 1926 applies to construction work; Subpart S, CFR 1926.800, applies to underground construction. USACE's EM 385-1-1, Safety and Health Requirements Manual, also applies. Some states have regulations that are more stringent than Federal regulations (see the California Tunnel Safety Orders). Contractors are responsible for the safety of the work, including temporary installations such as ventilation facilities and their operation and are therefore obliged to follow the law as enforced by OSHA. Contract documents do not usually contain specific requirements for ventilation, because such specific requirements might be seen as overriding applicable laws. In special cases, however, the tunnel designer may choose to incorporate specific ventilation requirements, supplementary to the applicable regulations. In such cases, the purpose is to make sure that the contractor is aware of the specific circumstances. By requesting submittals from the contractor on ventilation items, the owner/engineer can ascertain that the contractor does, indeed, follow regulations. Circumstances that may call for ventilation specification requirements include the following:

- An unusually long tunnel without intermediate ventilation shaft options.
- Certain potentially hazardous conditions, such as noxious or explosive gas occurrences, hot water inflow.
- Particularly extreme environmental conditions, such as very hot or very cold climatic conditions, where heating or cooling of air may be required.

- Circumstances where the ventilation system is left in place for use by a subsequent contractor or the owner; in these cases, the ventilation system should be designed almost as a part of the permanent system, rather than a temporary installation.

a. Purposes of underground ventilation. Underground ventilation serves at least the following purposes:

- Supply of adequate quality air for workers.
- Dilution or removal of construction-generated fumes from equipment and blasting or of gases entering the tunnel.
- Cooling of air—heat sources include equipment, high temperature of in situ rock or groundwater, high ambient temperature.
- Heating of air—sometimes required to prevent creation of ice from seepage water or from saturated exhaust air.
- Smoke exhaust in the event of underground fire-dust control.

Thus, designers of an underground ventilation system must consider the ambient and in situ temperatures, projected water inflow, potential for adverse conditions (gases), maximum number of personnel in the underground, types and number of equipment working underground, and methods of equipment cooling employed. In the permanent structure, ventilation provisions may be required for at least the following purposes:

- To bleed off air at high points of the alignment.
- To purge air entrained in the water, resulting, for example, from aeration in a drop shaft.
- For odor control and dilution of sulfide fumes in a sewer tunnel.
- To provide ventilation for personnel during inspection of empty tunnels.

These ventilation requirements often result in the use of separate permanent ventilation shafts with appropriate covers and valves.

b. Components of ventilation system. The principal components of a ventilation system are briefly listed below:

(1) *Fans.* Usually in-line axial or centrifugal fans are used. Fans can be very noisy, and silencers are usually installed. In a sensitive neighborhood, silencers are particularly important; alternatively, fans can be installed a sufficient distance away from the tunnel or shaft portals to reduce noise levels. Fans are designed to deliver a calculated airflow volume at a calculated pressure. With long vent lines, the required pressure may be too high for effective fan operation at one location (air leakage from vent lines also increase with increased differential pressure), and booster fans along the line are used. In the working areas, auxiliary fan installations are often required for dust control, ventilation of ancillary spaces, local air cooling, removal of gases or fumes, or other special services. When auxiliary fan systems are used, such systems shall minimize recirculation and provide ventilation that effectively sweeps the working places. Reversibility of fans is required to permit ventilation control for exhaust of smoke in case of fire.

(2) *Fan lines.* Rigid-wall fan lines made of steel ducting or fiberglass are sometimes used, mostly for exhaust; however, flexible ducting, made of flame retardant material, is more commonly used. Flexible ducting must retain an internal overpressure in order not to collapse. This requires reliable fan start control of all main and booster fans.

(3) *Scrubbers.* Excessive dust is generated from roadheader or TBM operation and is usually exhausted through scrubbers or dust collectors.

(4) *Ancillary ventilation structures.* These may include stoppings and brattices to isolate areas with different ventilation requirements or where no ventilation is required. In hot environments, cooling can be applied to the entire ventilation system, or spot coolers can be applied to working areas. Heaters can be required to prevent ice from forming at exhausts.

(5) *Monitors and controls.* These include air pressure and air flow monitors within the ducting or outside, monitoring of gases (methane, oxygen, carbon monoxide, radon, and others), temperature, humidity, and fan operation status. Stationary gas detectors located at strategic points in the ventilation system and at the face (e.g., mounted on the TBM) are often supplemented with hand-held detectors or sampling bottles. Signals would be monitored at the ventilation control center, usually at the ground surface, where all ventilation controls would be operated. Secondary monitors are often installed at the working area underground.

c. Design criteria. Typically, an air supply of at least 2.83 m^3/min (100 cfm) per brake horsepower of installed diesel equipment is required. Gasoline-operated equipment is not permitted, and diesel equipment must be provided with scrubbers and approved for underground operation. Mobile diesel-powered equipment used underground in atmospheres other than gassy operations shall be approved by MSHA (30 CFR Part 32), or shall be demonstrated to meet MSHA requirements. An additional air supply of 5.7 m^3/min (200 cfm) is required for each worker underground. Ventilation should achieve a working environment of less than 27 °C (80 °F) effective temperature, as defined in Hartman, Mutmansky, and Wang (1982). A minimum air velocity in the tunnel of 0.15 m/s (30 fpm) is usually required, but 0.5 m/s (100 fpm) is desirable. Air velocity should not exceed 3 m/s (600 fpm) to minimize airborne dust. For additional design criteria and methods, see SME Mining Engineering Handbook (1992) and ASHRAE Handbook (1989).

5-12. Surveying for Tunnels and Shafts

Technological advances in survey engineering have had a great influence on the design and construction of tunnels and shafts. From initial planning and integration of geotechnical and geographical data with topographical and utility mapping through the actual alignment and guidance of tunnel and shaft construction, survey engineering now plays a major role in the overall engineering and construction of underground structures. To benefit from these advances, survey engineers should be involved from the inception of planning through design and final construction. The results of these surveys would provide more cost-effective existing-conditions data, ranging from topographic mapping to detailed urban utility surveys; the use of appropriate coordinate systems tailored to meet the specific needs of the project; optimized alignments; more accurate surface and subsurface horizontal and vertical control networks properly tied to other systems and structures; precise layout and alignment of shaft and tunnel structures; and significant reduction in the impact of survey operations on tunnel advance rates.

a. Surveying and mapping tasks during planning.

(1) During the planning stage, the framework is constructed for all future project surveying and mapping efforts. Among the many important tasks to be performed at an early stage are the following:

- Select basic coordinate system and horizontal and vertical datums.
- Select or develop project-specific coordinate and mapping system.
- Provide tie-in with existing relevant coordinate and datum systems.
- Verify or renew existing monumentation and benchmarks.
- Develop specifications for required surveying and mapping activities.
- Procure existing map base and air photos as required.
- Supplement mapping as required for the purpose of planning.
- Prepare a Geographic Information System (GIS) base for future compilation of site data.

(2) In the United States, the standard reference for surveying is the North American Datum 1983 (NAD'83) for horizontal datum, and the North American Vertical Datum of 1988 (NAVD'88). State and local mapping systems are generally based on these systems, using either a Mercator or Lambert projection. Many localities employ, or have employed, local datums that must be correlated and reconciled. When specifying surveying or mapping work, it is necessary to indicate exactly which projection should be used.

(3) It is often appropriate, where greater accuracy is required, to develop a site-specific mapping system. Where the new structures are to be tied into existing facilities, the mapping base for the existing facilities can be extended. Often, however, it is better to modernize the system and remathematize the existing facilities as necessary.

(4) Topographic maps exist for virtually all of the United States, some of them in digital form. Depending on the age and scale of such mapping, they may be sufficient for initial planning efforts. More often than not, however, supplementary data are required, either because of inaccuracies in the available data or because of changes in land use or topography. Topographic and cultural data can be obtained from recent air photos or photos flown for the purpose, using photogrammetric techniques. Triangulation and traverses can be performed, using existing or new monuments and benchmarks, as part of the controls for photogrammetry and to verify existing mapping.

(5) Typically, reasonably detailed mapping in corridors 100 to 1,000 m (300-3,000 ft) wide are required along all contemplated alignments. This mapping should be sufficiently detailed to show natural and man-made constraints to the project. In urban areas, mapping of major utilities that may affect the project must also be procured, using utility owners' mapping and other information as available. At this time it may also be appropriate to secure property maps.

(6) Accurate topographic mapping is required to support surface geology mapping and the layout and projection of exploratory borings, whether existing or performed for the project.

(7) A computerized database, a GIS, is able to handle all of these types of information and to produce local maps and cross sections as required.

b. Surveying and mapping tasks during design.

(1) Mapping and profiling begun during planning must be completed during this phase. Also, all utilities must be mapped, as well as all buildings and other man-made features along the alignment. Property surveys must be completed to form the basis for securing the right-of-way.

(2) If not already available, highly accurate horizontal and vertical control surveys are required to tie down the components of the new facilities. The Global Positioning System (GPS) is helpful in providing precise references at low cost over long distances. The GPS is a satellite-based positioning system administered by the U.S. Air Force. When used in a differential mode in establishing control networks, GPS gives relative positioning accuracies as good as two ppm. GPS is also flexible, because line-of-sight is not required between points.

(3) The contract documents must contain all reference material necessary to conduct surveying control during construction. This includes generally at least the following:

- Mathematized line and grade drawings, overlain on profiles and topography from the mapping efforts. Designers will use a "working line" as a reference, usually the center or invert of the tunnel for a water tunnel, but some other defined line for transportation tunnels. All parts of the cross section along the tunnel are referenced to the working line.
- Drawings showing monuments and benchmarks to be used as primary controls. These should be verified or established for the project.
- Drawings showing existing conditions as appropriate, including all affected utilities, buildings, or other facilities.
- Interfaces with other parts of the project, as required.
- Specifications stating the accuracy requirements and the required quality control and quality assurance requirements, including required qualifications of surveyors. Where great accuracy is required, preanalysis of the surveying methodology should be required to demonstrate that sufficient accuracy can be obtained. Minimum requirements to the types and general stability of construction benchmarks and monuments may also be stated.

(4) Generally speaking, greater accuracy is required in urban areas with a great density of cultural features than in rural environments. Underground works for transportation, by their nature, require greater accuracy than most water conveyance tunnels.

(5) Benchmarks and monuments sometimes are located where they may be affected by the work or on swelling or soft ground where their stability is in doubt. Such benchmarks and monuments should be secured to a safe depth using special construction or tied back to stable points at regular intervals.

(6) Where existing structures and facilities may be affected by settlements or groundwater lowering during construction, preconstruction surveys should be conducted to establish a baseline for future effects. Such surveys should be supplemented by photographs.

c. Construction surveying and control.

(1) Except in rare instances, the contractor takes on all responsibilities for all surveying conducted for the construction work, including control of line and grade and layout of all facilities and structures. This permits the contractor to call on the surveyor's services exactly when needed and to schedule and control their work to avoid interferences. The owner or construction manager may

perform such work as is necessary to tie the work into adjacent existing or new construction. The owner or construction manager will also conduct verification surveys at regular intervals.

(2) The contractor's surveyor will establish temporary benchmarks and monuments as required for the work and is expected to verify the stability of these benchmarks.

(3) When a tunnel is driven from a portal, a baseline is typically established outside the portal and subsequently used as a basis for tunnel surveying. Line and grade is usually controlled by carrying a traverse through the tunnel, moving from wall to wall. This method will help compensate for surveying errors that can arise from lateral refraction problems resulting from temperature differences in the air along the tunnel walls. Rapid, high-precision survey work can be obtained using electronic levels and total-station equipment. High-precision gyrotheodolites can now provide astronomical azimuths with a standard deviation of 3 arc seconds, independent of refraction problems. This accuracy is rarely required as a standard for tunneling but is useful for verification surveys.

(4) Electromagnetic distance measuring instruments can provide accurate distance determinations between instrument and target very quickly and is the preferred method of distance measurement in tunnels.

(5) Shaft transfers have often been made using a plumb bob dampened by immersion in a bucket of water, with the vertical distance measured by a suspended tape. Two points at the shaft bottom must be established to create a baseline for tunneling. In a shaft of small diameter, the baseline thus transferred is short and therefore not accurate. In such cases, a backsight or foresight can be established by drilling a survey hole over the tail tunnel or the tunnel alignment. Such survey holes can also be used along the alignment for verification or correction in long tunnels.

(6) More modern shaft transfers are often done using an optical plummet. Vertical and horizontal shaft transfers using modern equipment, including total station, Taylor-Hobson sphere, precise level, and plummet, are accurate to depths of at least 250 m (800 ft).

(7) For a blasted tunnel, the tunnel face is marked with its center, based on laser light, and the blast layout is marked with paint marks on the face. The drill jumbo must be set accurately to ascertain parallelism of boreholes along the alignment and the proper angle of angled blastholes. An automated drill jumbo can be set up using laser light without marking the tunnel face.

(8) Modern TBMs are often equipped with semiautomated or fully automated guidance instrumentation (e.g., ZED, Leica, or DYWIDAG systems) that offers good advance rates with great precision. They require establishment of a laser line from a laser mounted on the tunnel wall. Laser beams disperse with distance and are subject to refraction from temperature variations along the tunnel wall. As a result, they must usually be reset every 250 m (800 ft) or less. For tunnels on a curve, lasers must often be reset at shorter intervals.

(9) Construction survey monuments are usually placed at a spacing of several hundred meters and at tangent points. These are sometimes made permanent marks. When placing the final, cast-in-place lining (if required), these monuments are also employed for setting the concrete forms precisely.

(10) Considering that TBMs provided with conveyor mucking systems sometimes advance at rates over 120 m/day (400 ft/day), it is evident that contractors must employ the best and fastest tools for advancing the survey controls along with the TBM in order not to slow down the advance. It is also clear that a small surveying error (or worse, a gross mistake) quickly can lead to a very costly misalignment. Thus, attention paid to the quality of the survey work and the tools used for surveying is well placed.

5-13. Construction Hazards and Safety Requirements

Underground construction has traditionally been considered a hazardous endeavor. Many years ago, this image was well deserved. Indeed, fatality rates during construction of classical tunnels such as the St. Gotthardt in Switzerland and the Hoosac in Massachusetts were extraordinarily high. In today's world, the frequency of accidents and the fatality rates for underground construction have approached those of other types of construction, partly because of a better understanding of causes of accidents and how to prevent them, and partly because of a greater degree of mechanization of underground works. This subsection explores common types of accidents in rock tunnels and cavern construction, their causes, and how to prevent them or to minimize their likelihood of occurrence. The potential for failures in the long term, during the operating life of tunnels, is dealt with in a later section.

a. Hazards related to geologic uncertainty. Contrary to many lay people's intuition, most tunnel accidents are not caused by rock fall or face collapse or some other geologically affected incident, but by some failure of equipment or human fallibility. Nonetheless, geologically affected failures or accidents occur, and on occasion such failures can be devastating and cause multiple fatalities. Typical accidents are discussed below.

(1) *Rock falls.* Rock falls result from inadequate support of blocks of rock that have the potential for falling or from insufficient scaling of loose blocks after a blast. Rocks can fall from the crown or the sidewalls of tunnels or from the face of a tunnel. The use of robots for installation of rock bolts or shotcrete over the muck pile after a blast greatly reduces the exposure of personnel. Rock falls also occur behind a TBM. A shielded TBM should not induce a sense of false security. Even a very small rock falling down a shaft becomes hazardous because of the high terminal velocity of the falling rock. Thus, particular attention must be paid to prevention of rock loosening around a shaft. Geologists and engineers sometimes venture out in front of the last installed ground support to map geology or to install instrumentation. More than one has been killed in this way, under a rock fall, and many have been injured.

(2) *Stress-induced failure.* Stress-induced failure occurs when in a massive or interlocking rock mass the stress induced around the underground opening exceeds the strength of the rock. Such events range in severity from delayed wedge fallouts in the crown or the sidewalls in soft rock, to popping or spalling, or violent rock bursts in hard and brittle rock.

(3) *Face or crown collapse.* This is relatively rare but can be very hazardous and costly when it occurs, as evidenced by case histories (see Box 5-1). These types of failure result either from encountering adverse conditions that were not expected and therefore not prepared for or from use of construction methods that were not suited for the adverse condition. The geological culprit is usually a zone of weakness, a fault zone with fractured and shattered rock, or soft and weathered material, often exacerbated by water inflow in large quantity or at high pressure.

(4) *Flooding or inrush of water.* Flooding or inrush of water is mostly an inconvenience, provided that adequate pumping capacity is available. The source of the water can be the interception of a pervious zone or a cavern with a substantial reservoir behind it, access to a body of water, or the breakage of a sewer or water line. In instances where the water does not naturally flow out of the tunnel, e.g., when tunneling from a shaft, adequate pumping capacity must be provided for safe evacuation. When tunneling in certain geothermally active terrains, inflow of scalding hot water can be a hazard. Large inflows of water have also occurred when tunnel construction accidentally intercepted an artesian well. When flooding brings with it large quantities of material, cohesionless sand or silt, or fault zone debris, several hundred feet of tunnel can be filled with debris or mud in a short time, causing personnel and machinery to be buried.

(4) *Gas explosions.* When gas explosions occur, they often cost a number of casualties. Examples include the San Fernando Water Tunnel in Sylmar, California, where a major methane gas explosion cost 17 lives. While recognized as a gassy tunnel, excessive amounts of gas were thought to have derived from a fault zone just ahead of the face. A Port Huron, Michigan, sewer tunnel was driven through Antrim Shale. During final lining installation, a methane explosion claimed 21 lives. More recently, a gas explosion in a tunnel in Milwaukee cost the lives of three people. The geological occurrence of methane gas is discussed in Section 3-7. Flammable and explosive gases in tunnels can (and should) be measured and monitored continuously. In some cases, automatic alarms or equipment shutdown is appropriate. Gas risks can be explored by probeholes ahead of the tunnel. Remedial actions include additional ventilation air, use of explosion-proof machinery, installation of gas-proof tunnel lining (used for the Los Angeles Metro), or predrainage of gas through advance boreholes.

(5) *Other harmful gases.* Other harmful gases may include asphyxiants as well as toxic gases (see Section 3-7):

- Nitrogen (asphyxiant) may derive from pockets in the strata.

- Carbon dioxide (asphyxiant, toxic above 10 percent) may derive from strata or dissolved in groundwater; it can result from acidic water reacting with carbonate rocks. Accumulates in depressions.

- Hydrogen sulfide (toxic) may derive from strata and groundwater, notably in volcanic terrains but also in connection with hydrocarbons. It is also present in sewer tunnels.

- Carbon monoxide (toxic) can also derive from the strata or the groundwater but is more often the result of fire.

Box 5-1. Case History: Wilson Tunnel Collapse

This highway tunnel on the Island of Oahu was driven with dimensions 10.4 m wide and 7.9 m high, 823 m long, through layered volcanics: basalt, ashes, clinker. Deep weathering was present on the leeward side of the range but not on the windward side. The tunnel was driven conventionally from the windward side, using full-face blasting as well as excavating tools. Ribs and lagging were used for ground support.

Driving through the relatively unweathered volcanics was uneventful. After advancing about 100 m full face into the weathered material on the leeward side, a collapse occurred some 25 m behind the face. Two weeks later, a second collapse occurred about 60 m behind the face, while the first collapse was not yet cleaned up. These two collapses did not result in casualties.

During reexcavation about 35 days after the first collapse, a third, disastrous collapse occurred, with five fatalities. Eighty meters of tunnel were buried in mud, and ground support and equipment were destroyed. Large cone-shaped depressions appeared at the ground surface.

The tunnel was eventually completed using an exploratory crown drift that acted as a drain, followed by multiple drifting. Bottom side drifts were completed first, and concrete foundations and walls placed to carry the arches constructed in crown drifts.

In this event, it appears that the contractor failed to modify his construction procedures as the ground characteristics changed drastically. Full-face excavation was not suited for this material, and the ground support was inadequate after a short period of exposure.

- Oxygen depletion can occur in soils and rocks due to oxidation of organic matter; if air is driven out of the soil into the tunnel, asphyxiation can result. Compressed-air tunneling has been known to drive oxygen-depleted air into building basements.

- Radon gas occurs mostly in igneous and metamorphic rocks, especially those that contain uranium. Radon changes into radioactive radon daughters that are harmful to the body.

Some gases, such as carbon monoxide and carbon dioxide, are heavier than air and therefore seek low points in underground openings. Workers have been asphyxiated going into shafts or wells filled with carbon dioxide. Other gases (methane) are lighter than air. Traps able to collect gases should be avoided.

(6) *Hazard reduction.* If a certain hazard exposure of a particular underground project were foreseeable, then provisions could be made to eliminate the hazard. It may be said, then, that geologic accidents or exposure to geologic hazards are the result of things unforeseen, i.e., lack of knowledge of conditions or things unforeseeable, i.e., uncertainty of behavior. These exposures also occur when danger signs are not noted, ignored, or misinterpreted. These findings form the basis for methods of hazard avoidance, as expressed in the following.

(a) Search for clues of geologic conditions that could be hazardous. Clues may be obtained from the general geologic environment—caverns in limestones, faulting and folding, deep weathering, volcanics, evidence of recent thermal action, hydrocarbons (coal, oil, or gas), unusual hydrologic regimes, hot springs, etc. Other clues should be searched for in the cultural records—records of tunneling or mining, construction difficulties of any kind, changes in hydrology, landslides, explorations for or production of oil or gas.

(b) During explorations, look for evidence of hazardous conditions. Based on the geologic environment and the initial search for clues of hazardous conditions, explorations can be focused in the most probable directions for confirmation of conditions and pinpointing hazardous locations. Tools are available to discover signs of hazards: airphoto and field mapping of geological features (faults, slides, hydrology), sampling of gases in boreholes (radon, methane, etc.), analysis of geologic structure and hydrology to extrapolate faults, discover gas traps, find anomalies of hydrostatic pressure to locate hydrologic barriers or conduits, etc.

(c) Establish plausible hazard exposure scenarios and evaluate the risks. If hazards are known with some certainty, they can be dealt with directly and in advance. For hazards of lower probability, prepare contingency plans

such that the hazards will be recognized in time during construction and remedial action can be taken. Provide means for dealing with expected (and unexpected) inflows of water.

(d) Provide for discovering hazards during construction: observe, map, and interpret rock as exposed during construction; measure concentrations of gases such as methane and radon; monitor water inflow, temperature, and other relevant parameters; drill probeholes ahead of the face to intercept and locate faults and pockets of water or gas.

(e) Remedial measures could include predrainage of water-bearing rock, grouting for strengthening and impermeabilization, modification of face advance methods (shorter rounds, partial-face instead of full-face advance), ground support methods (prereinforcement, spiling or forepoling, increasing ground support close to the face, etc.), shutting down equipment depending on methane concentration, and increasing ventilation to dilute gases. Mitigation of popping and bursting rock may include shaping the opening more favorably relative to stresses and installing (yielding) rock bolts and wire fabric.

(f) Maintain rigorous vigilance, even if everything seems to go right. Perform routine observations and monitoring of the face conditions as well as the already exposed rock surfaces. Do not walk under unsupported rock unless absolutely sure of its stability. Complacence and optimism do not pay, a rock fall can happen any time.

Knowledge of and preparedness for hazardous conditions should be embodied in a written plan for hazard control and reduction, as detailed as circumstances demand. The plan should be developed during exploration and design and incorporated as a part of construction contract documents. Safety plans and procedures, as well as safety training, are required for all work; special training is required for underground workers.

b. Hazards under human control.

(1) As already noted, many if not most tunnel accidents are at least in part under human control or caused by human action (or inaction). The examples described below are derived from the writer's personal knowledge and experience and are not hypothetical examples.

- Person falling from height (down shaft or from elevated equipment in tunnel or cavern).

- Person falling on the level (stumbling over equipment or debris left on floor, slipping on slick surfaces, exacerbated by often cramped conditions, limited space for movement, and poor lighting).

- Material falling from height (down the shaft, from equipment or vehicles, or from stacks or piles of material), including ice formed from seepage water.

- Interference with special tunneling equipment (person crushed by concrete lining segment erector or rock bolter, mangled in conveyor belt, or other moving piece of equipment—sometimes due to equipment malfunction, more often due to human error).

- Overstress of rock bolt or dowel or failure of anchorage during testing or installation, causing sudden failure of metal and a projectile-like release of metal (do not stand in the line of bolts or dowels tested).

- Moving-vehicle accidents (inspector run down by muck train or other vehicle, loco operator facing the wrong way hit by casing protruding down from the tunnel crown).

- Rock falls due to failure to recognize need for reinforcement.

- Electric accidents, electrocution (electrician failing to secure circuits before working on equipment, faults due to moisture entering electric equipment).

- Blasting accidents (flying rock, unexploded charges in muck pile, premature initiation, which could occur due to stray currents or radio activity, if using electric detonation).

- Fire and explosion other than from natural gas (electric fault as initiator, fumes from burning plastic, electric insulation, and other materials, burning of timber can result in loss of ground support, generation of carbon monoxide and other poisonous or asphyxiating gases).

- Atmospheric pollution due to equipment exhaust, explosives fumes, or dust generated from

- explosion, equipment movement, muck transport by cars or conveyor, dry or wet shotcrete application or TBM operation. Certain grouts have been known to release fumes during curing. Remedial measures: adherence to ventilation requirements, face masks.

- Heat exhaustion due to high temperature and humidity (preventable by adherence to regulations regarding thermal exposure).

- Excessive noise from drilling equipment, ventilator, or from blasting (ear plugs required).

(2) It is apparent that most of these types of accident or risk exposure could happen in many locations outside the tunnel environment. In fact, most of them are typical construction accidents. If they happen more commonly in the underground environment, it is for several reasons:

- Tunnels often provide very limited space for work and for people to move; thus people move slower and have a harder time getting out of the way of hazards.

- Poor lighting and limited visibility in the tunnel are other contributing factors.

- Often inadequate instruction and training of personnel in the detailed mechanics of tunneling make personnel inattentive to hazards and put them in the wrong place at the wrong time.

- Carelessness and inattention to safety requirements on the part of workers or supervisory personnel; unauthorized action on part of worker.

- Equipment failure, sometimes due to inadequate inspection and maintenance.

Prevention of accidents in tunnels and other underground works requires education and training of all personnel and rigorous and disciplined enforcement of safety rules and regulations during construction.

c. Safety regulations and safety plans.

(1) Safety of underground works other than mines is regulated by OSHA—the Occupational Safety and Health Act, 29CFR1926. Numerous other regulations govern various aspects of underground safety:

- The Internal Revenue Service 26CFR Part 181, Commerce in Explosives.

- 27CFR Part 55, administered by the Bureau of Alcohol, Tobacco and Firearms (both regulate manufacturing, trading, and storage as well as safekeeping of explosives).

- Department of Transportation 49CFR Part 173 and other Parts (regulate transportation of explosives).

- For DOD work, DOD 6055.9 - STD, Ammunition and Explosives Safety Standards, and DOD 4145.26 M, DOD Contractors Safety Manual for Ammunition and Explosives apply.

- The National Electric Code applies to all temporary and permanent electrical installations.

- MSHA - Mine Safety and Health Act, 30CFR Part 57 among other things defines and lists vehicles permissible underground.

(2) Among other documents that apply, American Congress of Government Industrial Hygienists' (ACGIH) Threshold Limit Values for Chemical Substances and Physical Agents in the Workroom Environment (1973) is important for ventilation of the underground. U.S. Environmental Protection Agency (EPA) regulations apply to handling and disposal of hazardous materials and contaminants.

(3) While, strictly speaking, the USACE is empowered to enforce its safety regulations on USACE projects, it is the practice to permit OSHA inspection and enforcement privileges. Where local regulations exist and are more stringent than OSHA, they are usually made to apply. An example of regulations exceeding OSHA in strictness is the State of California Tunnel Safety Orders.

(4) Contractors are obliged to follow all applicable Federal, state, and local laws and regulations and are generally responsible for safety on the job. Nonetheless, it is appropriate in the contract documents to reference the most important laws and regulations. It is also proper to require of the contractor certain standards and measures appropriate to the conditions and hazards of the project and for the USACE's resident engineering staff to enforce these standards and measures.

(5) For complicated or particularly hazardous projects, it is common to require the preparation of a Safety Analysis Report, in which all construction procedures are analyzed by the contractor, broken down to detailed subcomponents. The report also identifies all hazards, such that preventive and mitigating procedures can be developed and emergency measures prepared.

(6) For all projects, the contractor is required to prepare a full Safety Plan, subject to review and approval by the resident engineer, who will employ this plan for enforcement purposes. The act of preparing a project-specific Safety Analysis Report and Safety Plan, rather than using a standard or generic plan, will alert the contractor and the resident engineer to particular hazards that might not be covered by a standard plan, and will heighten the level of attention to safety provisions. Components of a typical Safety Plan may include the following types of items and other items as appropriate:

- Policy Statement: Elimination of accidents, no lost time due to accidents, safety takes precedence.

- References: Applicable laws and regulations.

- Responsibilities: Chains of command, administration and organization of safety program, authorizations required before commencing work, enforcement.

- Indoctrination and Training: Required training program for all, separate program for underground workers, required weekly toolbox safety meetings, requirements for posting information, etc.

- General Safety and Health Procedures: Housekeeping, material handling and storage, personal protective equipment, dealing with wall and floor openings, scaffolds, ladders, welding, flame cutting, electrical equipment, lock-out or tag-out procedures, motor vehicles, heavy equipment, small tools, concrete forms, steel erection, cranes and hoisting, work platforms, fire prevention and protection, sanitation, illumination, confined space entry, etc.

- Industrial Hygiene: Respiratory protection, noise, hazardous materials, submittal of Material Safety Data Sheets (MSDSs) and lists of hazardous chemicals present, hazards communication.

- Emergency Procedures: Detailed procedures for all types of emergencies, medical, fire, chemical spill, property damage, bomb threat, severe weather.

- Incident Investigation, Reporting, Record Keeping.

- Policy for Substance Abuse.

- Security Provisions.

(7) Additional provisions applicable to underground works include safety of hoisting, blasting safety, use of CO and CO_2 breathers (self-rescuers), which convert these gases to oxygen, access and egress control including emergency egress, safety inspection of exposed ground, storage of fuel underground, communications underground, monitoring of gases and dust in the tunnel, lighting and ventilation in the tunnel, and requirements to establish trained rescue teams.

(8) Depending on the number of people in the contractor's work force and the number of shifts worked, the contractor may be required to employ one or two persons who are fully dedicated safety officers. Likewise, one or more safety officers may also be required on the resident engineer's staff. Safety engineers are authorized to stop the work if a hazardous condition is discovered that requires work stoppage for correction. With proper cooperation and timely action, such work stoppages usually do not occur.

(9) Construction safety is serious business and must command the fullest attention of management personnel on all sides. An effective safety program relies on the following:

- Planning to avoid hazards.

- Detection of potential hazards.

- Timely correction of hazards.

- Dedication to the protection of the public and the worker.

- Active participation of all persons on the job.

- Dedicated safety staff.

5-14. Environmental Considerations and Effects

Many laws, rules, and regulations apply to underground construction. The National Environmental Policy Act, the

Clean Water Act, the Rivers and Harbors Act, the Endangered Species Act, and various regulations pertaining to historic and cultural resources are the major requirements that apply primarily to preconstruction phases. Regulatory programs that apply to construction include the following:

- Resource Conservation and Recovery Act (RCRA).

- Comprehensive Environmental Response, Compensation and Liability Act (CERCLA), also known as the Super Fund Act, including SARA Title III

- National Pollutant Discharge Elimination System (NPDES) permit program that is part of the Clean Water Act.

Satisfying the requirements imposed by these laws and regulations including associated permits are the focus of other documents and are not addressed in this manual. Accommodating environmental and permit requirements during construction involve little incremental cost or schedule disruption if the requirements are effectively addressed in planning, design, and contract documents. Early preconstruction work typically includes preparation of an Environmental Impact Statement (EIS). Design and construction constraints embodied in the EIS must be adhered to during design and construction.

a. Effects of settlements and ground movements.

(1) Ground movements and settlements occur either as a result of elastic or inelastic relaxation of the ground when excavation relieves in situ pressures or as a result of groundwater lowering. Lowering the groundwater table can result in compaction or consolidation of loose or soft overburden. Removal of fines by seepage water or via dewatering wells can also result in settlements. Gross instability and collapse of tunnel face (or shaft bottom) also cause ground surface depressions.

(2) Tunnels and shafts in rock, when properly stabilized, usually do not result in measurable ground settlements. On the other hand, ground movement control is a major issue for tunnels and excavations in soil in urban areas, especially if below the groundwater table.

(3) When damaging settlements are deemed possible for a rock shaft or tunnel project (e.g., shaft through overburden, effect of dewatering), the following provisions should be taken:

- Preconstruction surveys with photos or video, documenting existing conditions.

- Contract requirements to limit or eliminate effects that can cause settlements.

- Monitoring of construction performance (measurements of ground motions, settlements of buildings, groundwater level, etc.).

- Provisions to pay for damage, if any (cost sometimes to be borne by the contractor).

(4) In general, contractual provisions should be devised that will encourage the contractor to conduct his work with a minimum of ground motions.

b. Groundwater control and disposal.

(1) Groundwater levels should be maintained during construction, if practicable, to avoid a number of risks including unexpected ground settlement, entrainment of pollutants from underground tanks or other sources, affecting surface water systems, and water quality concerns associated with disposal. If shafts are required for tunnel access, methods of shaft sinking should be adopted that do not require aggressive pumping to create a cone of depression prior to installation of the lining.

(2) In many cases, excessive infiltration of groundwater into tunnels and shafts during or after construction is unacceptable because wells owned and operated by private persons or public agencies may be seriously affected by lowering of the groundwater. Concern for the natural environment, including existing vegetation, springs, and creeks, can require tight control of water infiltration both during construction and operations. Monitoring of the surface hydrology as well as observation wells is often required to ascertain effects of tunneling and show compliance with performance restrictions. If unacceptable effects are found, remedial action may be required.

(3) Effective management of tunnel seepage includes discharge to onsite settling ponds or tanks of sufficient capacity to reduce suspended solids to acceptable levels before discharging tunnel seepage into a storm water system or surface stream. The water management system should also have a means of detecting and removing petroleum hydrocarbons prior to discharge. This can be accomplished through an oil-water separator or passing the discharge through oil-sorbent material in combination with a settling basin or pond.

(4) Widely accepted standards for hydrocarbon concentrations in discharged water are "no visible sheen" and no more than 15 parts per million (ppm). The acceptable

pH range for discharged water often is between 6.0 and 9.0, although some states or localities may have narrower limits. Standards or policies established during the design should be incorporated in the contract requirements so that compliance costs will be reflected in bids.

(5) Conflicts with agency staff and landowners will be minimized if contractors clean up leaks and spills in the tunnel, conduct grouting and shotcrete activities so as to prevent highly alkaline water from leaving the site, and have emergency equipment and materials on hand to effectively manage water that may become contaminated by a construction emergency.

(6) Frequent, systematic site inspections to evaluate construction practices are effective in documenting conditions and in identifying corrective action that must be taken. Corrective actions can also be tracked and closed out after being implemented. Documentation includes photographs and water quality data from onsite ponds and discharge.

(7) Leakage from underground tanks and pipelines, leachate from landfills, or contamination from illegal dumping or surface pits are a few of the conditions that may be encountered during tunneling. Preconstruction surveys can provide an indication if current or past land uses are likely to have contaminated areas where the tunnel will be constructed. In such cases, the designer should anticipate possible adverse effects on tunnel linings as well as measures for proper management and disposal. In the extreme, aligning the tunnel to avoid such areas may be the most cost-effective solution. Avoidance also limits the potential long-term liability that is associated with handling and disposing of contaminated solids and liquid wastes.

(8) Unexpected contamination can occur where underground fuel tanks have been in use for many years. Overfilling and leaks can result in high concentrations of gasoline and fuel oil, which present a hazard to work crews as well as high costs for disposal. Other potential sources of contamination include commercial cleaning shops and abandoned industrial facilities.

(9) The environmental hazard and liability are often minimized by contracting, in advance of construction, with a firm that will provide emergency response. This would include services to contain contamination, test water and soils to determine the types and concentrations of contaminants, provide advice on possible contamination sources, and advise and assist in proper disposal. Alternatively, contamination could be removed before tunneling.

(10) On occasion, a water supply tunnel will traverse a region of brackish groundwater or brine or water containing other unacceptable chemicals. Here, a nominally watertight lining must usually be provided to minimize infiltration. In the case of sewer tunnels, exfiltration can contaminate surrounding aquifers. Sewer pipes and tunnels must usually meet water tightness requirements laid down by local authorities.

c. Spoil management.

(1) Disposal of material removed from tunnels and shafts is often the source of considerable discussion during the environmental planning phase.

(2) In rural areas, tunnel muck can often be disposed of onsite without adversely affecting surface or ground water. In urban areas, it may have to be transported to locations well removed from the point of generation. Except for special circumstances, tunnel muck in the urban environment is usually disposed of by the contractor, who is obliged to follow applicable regulations.

(3) Total petroleum hydrocarbon concentrations in soil, muck, or sediment can restrict management and disposal options. A widely accepted criterion for total petroleum hydrocarbon concentration is 100 ppm. Muck up to this concentration can be disposed of onsite, whereas muck with higher concentrations requires special disposal. The requirements for a specific project location should be determined during the design and included in the contract documents. The costs for managing muck that exceeds criteria are typically high and can be an inducement for contractors to carefully handle fuels and oils. It is often thought that tunnel muck produced by a TBM is useful as concrete aggregate. TBM muck, however, consists of elongated and sharp-edged pieces of rock, unsuitable for concrete aggregate. Recrushing generally does not help. TBM muck, however, is useful as road fill.

(4) The size and shape of spoil piles is frequently an issue once the location has been determined. Maximum pile height and sideslope grade, desirable configurations or shapes, and permanent ground cover would be determined based on the specific of each project.

(5) RCRA, CERCLA, NPDES, and state rules and regulations can involve special management techniques. Waste water and spoil that has naturally high heavy metal content, has high levels of radioactive isotopes, or is contaminated by some action or facility owned by others could produce harmful leachate. The potential for these to

occur depends on the location and nature of the project. Construction monitoring to detect, characterize, and properly manage the disposal of excess material should be conducted to document that spoil is being properly handled.

d. Waste Waters. Equipment and construction may generate "process" waste waters that require Federal or state permits to discharge into surface waters. Federal and state regulations may require a permit to discharge TBM cooling water, wash water from scrubbers, waste from onsite treatment processes, pipe flushes and disinfectants, or other nonstorm waters. Regulatory requirements are determined from the particular type of nonstorm water discharged, even if it meets the highest standard of quality. The contract documents should indicate which waters cannot be discharged into surface drainage if permits cannot be acquired prior to contract award.

e. Control of fugitive dust.

(1) The 24-hr and annual National Ambient Air Quality Standards (NAAQS) established for dust particles 10 μm are maximum 150 $\mu g/m^3$ and 50 $\mu g/m^3$ of air, respectively. Such particles tend to become trapped in lungs and pose a long-term hazard. Larger particles are not always regulated by a quantitative standard, but can result in regulatory action if there are complaints. Stringent dust control standards may apply to construction fugitive dust emissions for projects located in air sheds that do not meet the NAAQS for particulates.

(2) Confining dust to a construction site is difficult if the site is small, the rock tends to produce a large percentage of fines, and the contractor's muck handling method involves a number of transfers, or there is heavy traffic on unpaved roads. Raising the moisture content of muck with water in combination with shrouds or other devices is an effective measure to confine dust in the work area. This frequently involves situating spray nozzles at vent outlets, along conveyor transfer points, on stackers, and on temporary muck piles that will be loaded and transported to the disposal area. Paved construction roads are also an effective dust control measure. Establishing a criterion for "no visible dust" outside the construction boundary and leaving the means and methods to contractors may not result in acceptable dust control.

f. Storm water runoff and erosion control.

(1) A general NPDES permit to discharge storm water from construction sites larger than 5 acres was published by EPA in the Federal Register, September 9, 1992. The permit requires EPA to be notified when construction is started and completed but requires no other routine filings. A storm water pollution prevention plan (SWPPP), various certifications, and periodic site inspections are to be maintained at the construction site. The plan must be site specific and address techniques to divert overland flow around disturbed areas, stabilize slopes to prevent erosion, control runoff from disturbed areas so that sediment is trapped, prevent mud from being tracked onto public roads, and properly store and handle fuel, construction chemicals, and wastes.

(2) The SWPPP must satisfy standards contained in the regulations. Contractually, this could be accomplished by setting a performance standard or by developing a detailed plan that the contractor must implement. The former approach enables contractors to apply their expertise and knowledge of the area and relieves designers of predicting a contractor's requirements for temporary facilities. It does, however, put the owner at risk if the contractor does a poor job of planning or executing the plan.

(3) The latter approach gives the owner much more control over compliance. The procurement documents would contain the plan and a copy of the filed Notice of Intent, as well as a partially completed notice of termination, which the contractor would complete and file at the end of the job. The contractor could make changes in the storm water plan, but only after proposing them in a form that could be incorporated into the plan and receiving written approval from the owner.

g. Noise and vibration.

(1) Incorporated urban areas typically have noise and vibration ordinances that may apply to tunneling. These would be satisfied by surrounding noise sources in acoustical enclosures, erecting sound walls, limiting noise-generation activities to certain times of the day, or by using equipment designed to achieve reduced noise levels.

(2) Acceptable construction noise levels at a sensitive receptor (e.g., dwelling, hospital, park) may be established for day and night by state or local agencies. Some degree of noise monitoring prior to and during construction is advisable. An integrating precision sound level meter that provides maximum, minimum, and equivalent (average) noise outputs is appropriate. A typical day and night noise level limit for rural areas is 55 dBA and 45 dBA, respectively. For residential areas in cities, acceptable noise levels would typically range between 65 dBA and 75 dBA, with higher levels for commercial areas.

(3) Vibration and air-blast noise are usually associated with blasting, an activity that is readily controlled to achieve applicable standards. Monitoring and control of blasting vibrations are discussed in Section 5-1-e.

h. Contingency planning.

(1) Underground construction can encounter unexpected conditions and involve incidents that can release pollutants into the environment. Developing strategies to accommodate the types of events that could result in polluting water and soil is an effective method to reduce impacts and liability. Examples of pollution-causing incidents include a massive loss of hydraulic fluid in the tunnel, large inflow of groundwater, rupture of diesel fuel tank on the surface, vehicle accident involving diesel spill, fire, and the release of hazardous construction chemicals.

(2) Advance planning strategies include proper storage of fuels and chemicals, secondary containment, good housekeeping, training for all persons in corrective actions during incidents, bolstered by periodic discussion in tool box sessions, stockpiling response kits and containers to initiate proper cleanup, and having a contract in place with qualified emergency response personnel.

(3) The requirements contained in 40 CFR 112, which requires a spill prevention, control, and countermeasures plan if certain oil storage limits are exceeded, provides a good model on which to start contingency planning.

5-15. Contracting Practices

A principal goal in preparing contract documents is to achieve a contract that will yield a fair price for the work performed, acceptable quality of the work, and a minimum of disputes. A number of different contract provisions are employed to achieve these goals. Several of these clauses serve to minimize the need for bidders to include large contingencies in their bid to make up for the uncertainty often associated with underground works. The USACE employs a large number of standard provisions and clauses in the preparation of contract documents. Many of these can be used for underground works as they are, but a number of them require modifications to make them apply to the particular working conditions and project requirements of underground works. Each clause contemplated for use should be read carefully and modified as required. As an example, concrete placement for a final lining is very different from concrete placement for surface structures. Specifications for initial ground support, as well as for tunnel and shaft excavation, must usually be tailored to conditions for the particular tunnel.

a. Clauses. A number of clauses are of particular use in underground works; these are discussed briefly in the following.

(1) *Differing site conditions.* The differing site conditions clause is now a standard in most contracts, including those funded by Federal moneys. The clause provides that the contractor is entitled to additional reimbursement if conditions (geologic or other) differ from what is represented in the contract documents and if these conditions cause the contractor to expend additional time and money.

(2) *Full disclosure of available subsurface information.* All available factual subsurface information should be fully disclosed to bidders, without disclaimers. This is usually achieved by making all geotechnical data reports available to the bidders. In addition, the designer's assessment as to how the subsurface conditions affected the design and the designer's interpretation of construction conditions are usually presented in a GDSR. This report is usually made a part of the contract documents. This report should carefully define what the contractor can assume for his bid, which risks are to be borne by the owner and which by the contractor, and what will be the basis for any differing site conditions claims. The use of the GDSR as a baseline document is not at this time a standard practice for USACE projects.

(3) *Contract variations in price.*

(a) When a construction contract is relatively small and the work is well defined with little chance of design changes, and when the geology is well defined, a lump sum type of contract is often appropriate. Most often, however, underground construction contracts are better served by another type of contract in which certain well-defined parts of the work are paid for in individual lump sums, while other parts are paid for on a unit price basis. This permits equitable payment for portions of the work where quantities are uncertain.

(b) As an example, the required initial ground support in a rock tunnel is not known with certainty until conditions are exposed in the tunnel. It is common practice to show three or more different ground support schemes or methods, suitable for different rock quality as exposed. For each scheme, the contractor bids a unit price per foot of tunnel. The designer provides an estimate of how much of each type of ground support will be needed; this estimate provides the basis for the contractor's bid. The contractor will then be paid according to the actual footage of tunnel where each different ground support scheme is required.

(c) Other construction items that may be suited for unit pricing include the following as examples:

- Probehole drilling, per meter (foot).
- Preventive or remedial grouting, per meter (foot) of grout hole, per hookup and per quantity of grout injected.
- Supplementary payments if estimated water inflow is exceeded, possibly on a graduated scale.
- Different payment for excavation of different rock (soil) types if excavation efforts are expected to be significantly different and quantities are unknown.
- Payment for stopping TBM advance (hourly rate) if necessary to perform probehole drilling or grouting or to deal with excessive groundwater inflow or other defined inclement.

(d) When preparing a bid schedule with variable bid items, it is wise to watch for opportunities where the bidder could unbalance the bid by placing excessive unit prices on items with small quantities. Each quantity should be large enough to affect the bid total. In some cases, unit prices are "upset" at a maximum permitted price to avoid unbalancing.

(e) There is usually a standard clause providing for adjustment to unit prices if changes in quantities exceed a certain amount, usually 15 or 20 percent. Depending on the certainty with which conditions are known, some or all of the unit prices discussed here may be excluded from this clause.

b. Other contracting techniques.

(1) *Dispute Review Board.*

(a) Legal pursuit of disputes arising from contractor claims are expensive, tedious, and time-consuming. Disputes also bring about adversary relations between contractor and owner during construction. Dispute Review Boards (DRBs) go a long way toward minimizing or eliminating disputes by fostering an atmosphere of open disclosure and rapid resolution during construction, when the basis for any claims is still fresh in memory. The use of DRBs is extensively described in ASCE (1994).

(b) The typical DRB consists of three members—one selected by the contractor, one by the owner, and one by the first two members, all subject to approval by both parties and all experienced in the type(s) of work at hand and in interpreting and understanding the written word of the contract. The DRB members must have no vested interest in the project or the parties to the construction contract other than their employment as DRB members. The DRB usually meets every 3 months to familiarize themselves with the project activities. Claims between the contractor and the owner that have not been resolved will be brought before the DRB, who will render a finding of entitlement and, if requested, a finding of quantum (dollars, time). These findings are recommendations only and must be agreed to by both parties. The contractor will still have legal recourse, but the findings of the DRB are admissible as evidence in court.

(c) Because the DRB members have no monetary interest in the matter (other than their DRB membership), and because they are usually seasoned and respected members of the profession, their findings are almost always accepted by the parties, and the dispute is resolved in short order, while the matter is fresh and before it can damage relations on the job site.

(2) *Escrow of bid documents.* DRBs are usually recommended in conjunction with the use of escrowing of bid documents (ASCE 1994). A copy of the contractor's documentation for the basis of the bid, including all assumptions made in calculation of prices, is taken into escrow shortly after the bid. At the time of escrow, the documents are examined only for completeness. The documents can be made available to the parties of the contract and the DRB if all parties agree. By examining the original basis for the bid, it is often found easier to settle on monetary awards for contract changes and differing site conditions.

(3) *Partnering and shared risk.*

(a) The USACE introduced the concept of partnering in 1989. It includes a written agreement to address all issues as partners rather than as adversaries. Contracting issues involving risk sharing and indemnification may be discussed within the partnering agreement. This requires both training and indoctrination of the people involved. Partnering also includes at least the following components:

- A starting, professionally guided workshop of 1 or 2 day's duration, where the emphasis is on mutual understanding and appreciation and development of commitments to work together with team spirit.

- Continuing periodic partnership meetings, usually addressing job problems but structured to approach them as partners rather than antagonists; a professional facilitator usually leads these meetings.

(b) Experience with partnering has been good, and it is felt that this device has reduced the number of disputes that arrive in front of the DRB. Partnering will not resolve honest differences of opinion or interpretation but will probably make them easier to resolve.

(4) *Prequalification of contractors.* For large and complex projects requiring contractors with special expertise, it is common to prequalify contractors for bidding. For USACE projects this is rarely done. Some time before contract documents are released for bidding, an invitation is published for contractors to review project information and submit qualifications in accordance with specific formats and requirements prepared for the project. Only those qualifying financially as well as technically will be permitted to submit bids on the contract. Prequalification can apply to the contracting firm's experience and track record, qualifications of proposed personnel, and financial track records.

5-16. Practical Considerations for the Planning of Tunnel Projects

For many tunnels, size, line, and grade are firmly determined by functional requirements. This is true of most traffic tunnels as well as gravity sewer tunnels. For other types of tunnels, these parameters can be selected within certain bounds. A summary is presented below of a few practical hints for the planning of economical tunnels.

a. Size or diameter of tunnel. Hard-rock TBMs have been built to sizes over 10 m in diameter (33 ft); span widths for blasted openings are restricted only by rock quality and rock cover. For rapid and economical tunneling of relatively long tunnels, a diameter of about 4.5 m (15 ft) or larger (3.5-m (11.5-ft) width for horseshoe shape) is convenient. This tunnel size permits the installation of a California switch to accommodate a 1.07-m (42-in.) gage rail, which allows passing of reasonably sized train cars. Smallest tunnel diameter or width conveniently driven by TBM or blasting is about 2.1-2.4 m (7-8 ft). Pilot or exploratory tunnels are usually driven at a size of 2.4-3 m (8-10 ft), depending on length. Smaller tunnels can be driven by microtunneling methods.

b. Shaft sizes. Shafts excavated by blasting should be at least 3-3.5 m (10-12 ft) in size; the maximum size is not limited by the method of excavation. Blind drilling with reaming using triple kelly is limited to about 7.5 m (25 ft) at 80 m (270 ft) depth. Blind drilling using reverse circulation can produce shafts to a diameter of more than 6 m (20 ft), depending on depth and rock hardness. The maximum depth achieved using blind drilling through hard rock is in excess of 1,000 m (3,300 ft), with a drilled diameter of about 3 m, and finished diameter of the steel casing of 1.8 m (6 ft). Raise drilling is currently limited to about 6 m (20 ft) in diameter, depending on rock strength and hardness.

c. Grade or inclination of tunnel. With rail transport in the tunnel, a grade of 2 percent is normal, and 3 percent is usually considered the maximum grade. Higher grades—up to more than 12 percent—can be used with cable hoisting gear or similar equipment. Rail transport usually occurs at a maximum velocity of 15 mph. Rail transport has limited flexibility but is economical compared with rubber-tired transport for longer (> about 1.6 km (1 mi)) tunnels. Rubber-tired equipment can conveniently negotiate a 10-percent grade, but up to 25 percent is possible. The usual maximum speed is about 25 mph. For conveyor belts, a grade of 17 percent is a good maximum, though 20 percent can be accommodated with muck that does not roll down the belt easily. Depending on belt width, the maximum particle size is 0.3-0.45 m (12-18 in.). Most belts run straight, but some modern belts can negotiate large-radius curves. Pipelines using hydraulic or pneumatic systems can be used at any grade but are rarely used. Usually, shafts shallower than 30 m (100 ft) employ cranes for hoisting; a headframe is used for deeper shafts. Vertical conveyors are used for muck removal through shafts to depths greater than 120 m (400 ft).

d. TBM versus blasting excavation of tunnels. No hard and fast rules apply on the selection of excavation methods for tunneling. The economy of TBM versus other mechanical excavation versus blasting depends on tunnel length, size, rock type, major rock weaknesses such as shear zones, schedule requirements, and numerous other factors. Cost and schedule estimates are often required to determine the most feasible method. On occasion, it is appropriate to permit either of these methods and provide design details for both or all. From a recent survey of USACE tunnels, all tunnels greater in length than 1,200 m (4,000 ft) were driven with TBM, and all under that length were driven using blasting techniques. Tunnels driven by USACE also include roadheader-driven tunnels in Kentucky, West Virginia, and New Mexico, all about 600 m (2,000 ft) long.

e. Staging area. Where space is available, the typical staging area for a tunnel or shaft project can usually be

fitted into an area of about 90 by 150 m (300 by 500 ft). An area of this size can be used for space-planning purposes. If space is restricted, for example in an urban area, there are many ways to reduce the work area requirements, and many urban sites have been restricted to areas of 30 by 60 m (100 by 200 ft) or less. Such constraints cause contractor inconvenience, delays, and additional costs. If contaminated drainage water must be dealt with, the water treatment plant and siltation basin must also be considered in the estimate of work area requirements.

Chapter 6
Design Considerations

6-1. Fundamental Approach to Ground Support Design

a. Underground design must achieve functionality, stability, and safety of the underground openings during and after construction and for as long as the underground structure is expected to function. There is no recognized U.S. standard, practice, or code for the design of underground structures. Many designers apply codes such as ACI's Codes and Practices for concrete design, but these were developed for structures above ground, not for underground structures, and only parts of these codes apply to underground structures.

b. Designers often approach tunnel design by searching for modes of failure that can be analyzed (e.g., combined bending and compression in a lining), then apply them to more-or-less realistic but postulated situations (loading of a lining). While bending and compression are applicable failure modes for linings, many other modes of failure must be analyzed. In principle, all realistic modes of behavior or failure must be defined; then means by which these can be analyzed and mitigated must be found.

c. Failure modes are modes of behavior that could be considered unacceptable in terms of hazard, risk to cost or schedule during construction, environmental effect, or long-term failure of function. For underground structures, failure of function means different things for different kinds of structures: a certain amount of leakage in an urban highway tunnel might be a failure of function, while for a rural water conveyance tunnel such leakage might be perfectly acceptable.

d. The five basic design steps are outlined below:

(1) The functional requirements are defined in a broad sense. They include all hydraulic and geometric requirements, ancillary and environmental requirements and limitations, logistics, and maintenance requirements.

(2) Collect geologic and cultural data including all information required to define potential failure modes and analyze them, field and laboratory data, and cultural data to define environmental effects and constraints. These data may include ownership of right-of-way, the possibility of encountering contaminants, and sensitivity of structures to settlements.

(3) Determine plausible and possible failure modes including construction events, unsatisfactory long-term performance, and failure to meet environmental requirements. Examples include instability problems or groundwater inflow during construction, corrosion or excessive wear of ground support elements, excessive leakage (in or out), and settlements that may cause distress to adjacent existing structures.

(4) Design initial and final ground supports. Initial support includes all systems that are used to maintain a stable, safe opening during construction. Final supports are those systems that need to maintain a functional opening for the design life of the project. Initial supports may constitute a part of the final supports, or they may be the final support (e.g., precast segmental liner installed behind a TBM).

(5) Prepare contract documents. This is the synthesis of all design efforts and may include provisions to modify construction procedures based on observations. The contract documents also contain all information necessary for a competitive bidding process, and means to deal with claims and disputes.

e. The following subsections describe functional requirements of tunnels and shafts, typical and not so typical modes of failure of tunnels and shafts, including corrosion and seismic effects. Selection and design of initial ground support are described in Chapter 7, and final lining selection and design in Chapter 9.

6-2. Functional Requirements of Tunnels and Shafts

Most USACE tunnels are built for water conveyance, either for hydropower, fresh water transport, or flood control. Underground hydraulic structures may include drop and riser shafts, inclines, tunnels, intakes, outlets, intersections, bifurcations, energy dissipators, venturi sections, sediment control, surge chambers, gates, and valves.

a. Types of flow in underground hydraulic structures.

(1) Flow in underground hydraulic structures will be either open-channel flow or pressurized flow. Pressurized flow is usually under positive pressure, but negative pressures can also be encountered.

(2) If it is desired to maintain gravity flow conditions in a tunnel, then the size and grades must be designed to accomplish this. Usually, the variable flow quantities and input pressures (minimums and maximums) are given and cannot be adjusted. In some cases, geologic conditions may limit adjustments to grade. On the other hand, it may be desired to generate pressurized flow, for example in a hydropower intake tunnel to spin the turbines, in which case size and grade are selected for that purpose. Trade-offs can be made between size and grade to determine whether pressurized or gravity flow will occur and which is more desirable for a specific facility.

(3) Short tunnels of 100 m (330 ft) or less can be driven level, but longer tunnels are usually constructed at a minimum slope of 0.0001 (0.01 percent) to facilitate drainage.

(a) *Open-channel (gravity) flow hydraulic structures.* In open-channel flow, the water surface is exposed to the atmosphere. This will be the case so long as the rate of flow into the structure does not exceed the capacity as an open channel. For a gravity flow tunnel with multiple input sources or changes in cross section or grade, various points along the alignment must be analyzed to ascertain the flow volume and velocity to make certain that this condition is met. Hydraulic jumps can form within open channels if the slope of the channel is too steep or the outlet is submerged. If the hydraulic jump has sufficiently high energy, damage to the structure can result. This condition should be avoided.

(b) *Pressurized structures.* When the flow rate exceeds the open-channel capacity of the structure, it becomes pressurized. This may be a temporary condition or may be the normal operating configuration of the facility. Cavitation occurs in flowing liquids at pressures below the vapor pressure of the liquid. Because of low pressures, portions of the liquid vaporize, with subsequent formation of vapor cavities. As these cavities are carried a short distance downstream, abrupt pressure increases force them to collapse, or implode. The implosion and ensuing inrush of liquid produce regions of very high pressure, which extend into the pores of the hydraulic structure lining. Since these vapor cavities form and collapse at very high frequencies, weakening of the lining results as fatigue develops and pitting appears. Cavitation can be prevented by keeping the liquid pressure at all points above the vapor pressure. The occurrence of cavitation is a function of turbulence in the water flow and increases with tunnel roughness and flow velocity.

b. Hydraulic controls.

(1) Hydraulic controls are placed in a flow channel to regulate and measure flow and to maintain water levels upstream of a section. Over the full length of a tunnel, a variety of flow conditions may exist in each of the segments. Discharge and flow depth are determined by the slope, geometry, and lining of a tunnel and by the locations of hydraulic controls such as gates, weirs, valves, intakes, and drop structures. Within each segment of a tunnel, the segment inlet or outlet can serve as the control section. Inlet control will exist when water can flow through a tunnel segment at a greater rate than water can enter the inlet. Headwater depth and inlet geometry determine the inlet discharge capacity. Segments of a tunnel operating under inlet control will generally flow partially full.

(2) Outlet control occurs when control sections are placed at or near the end of a tunnel segment and water can enter the segment at a faster rate than it can flow through the segment. Tunnel segments flowing under outlet control will flow either full or partially full. The flow capacity of a section flowing under outlet control depends on the hydraulic factors upstream of the outlet.

(3) Weirs are one form of hydraulic control commonly used to regulate and measure flow in open channels. Many variations in weir design exist, most of which are accompanied by their own empirical equations for the design of the weir. Weir equations and coefficients are found in most textbooks dealing with open-channel flow.

c. Transient pressures.

(1) Transient pressures are a form of unsteady flow induced whenever the velocity of moving water in a closed conduit is disrupted. Causes include changes in valve or gate settings, pump or power failures, lining failures, and filling of empty lines too quickly. One type of transient flow is known as water hammer. This phenomenon is a significant design consideration in water tunnels because of the structural damage that can occur with excessive high or low pressures. There are many other types of transient flows in tunnels that can be caused by unequal filling rates at different locations along the tunnel: air entrainment, air releases, and hydraulic jumps. For structural analysis, lower safety or load factors are used when designing for transient pressures.

(2) Transient pressure pulses arise from the rapid conversion of kinetic energy to pressure and can be

positive or negative depending on position with respect to the obstruction. Pressure pulses will propagate throughout a tunnel or pipe system being reflected at the ends and transmitted and reflected where cross sections change. The magnitude and propagation speed of a pressure pulse are determined by the elastic characteristics of the fluid and the conduit and the rate at which the velocity is changed. All other factors being equal, the more rapid the velocity change, the more severe the change in pressure.

(3) Transient pressures are managed by careful placement of surge tanks, regulated valve closure times, surge relief valves, or a combination of these methods.

(4) Transient pressures should be analyzed for each and every tunnel by the hydraulic engineering staff for use in the design of pressure tunnels. For preliminary use, a transient pressure 50 percent higher than the operating design pressure is often used.

d Air relief.

(1) Air that occupies an empty or partially filled tunnel can become trapped and lead to operating difficulties ranging from increased head loss and unsteady flow to severe transients and blowouts. Air can enter a tunnel system by entrainment in the water at pump inlets, siphon breakers, drop structures, and hydraulic jumps. It can also form when pressure and temperature conditions cause dissolved air to be released.

(2) Engineering measures to reduce air entrainment include thorough evaluation of drop structures under all foreseeable flow conditions, elimination of hydraulic jumps by reducing channel slopes or other means, and dissipation of flow vortices at inlets.

(3) Air entrapment can lead to increased head losses caused by a constricted flow cross section, and more significantly, severe transient pressures when trapped air is allowed to vent rapidly. Air entrapment at changes in tunnel cross sections are avoided by matching tunnel crown elevations rather than matching the inverts. Vents to the ground surface frequently are used for air pressure relief.

e. Roughness.

(1) The roughness of a tunnel lining relative to its cross-sectional dimensions is fundamental to the efficiency with which it will convey water. Tunnel excavation methods, geometry, and lining type affect flow capacity and play important structural and economic roles in water tunnel design. The allowable velocities in different kinds of water tunnels are restricted by potential cavitation damage depending on the liner material used, sediment deposit, and flushing characteristics.

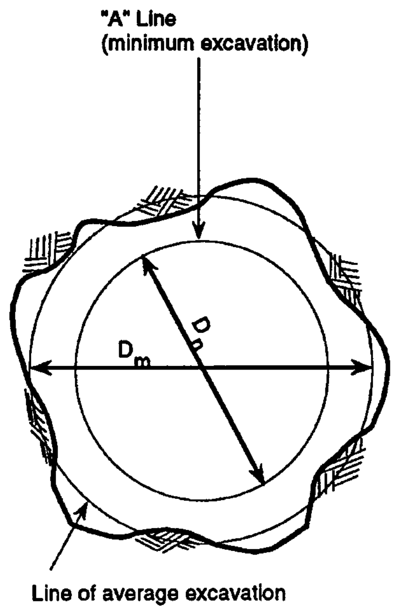
(2) The determination of tunnel friction factors for use with the Manning or Darcy-Weisbach flow equations is complicated by changes in flow depth, irregular channel geometries, and the wide range of roughnesses that occur when multiple lining types are used. Friction coefficients for the Manning and Darcy-Weisbach equations are each affected, but to different degrees, by changes in velocity, depth of flow, lining material, tunnel size, and tunnel shape. The Darcy-Weisbach approach is technically the more rigorous of the two equations; however, the Manning equation survives in practice because of its reasonable accuracy as an approximation for typical tunnel sizes and its relative simplicity.

(3) In practice, fluid velocities are limited so that turbulent conditions and the possibility of damage to the structure are limited. Velocities of less than about 3 m/s (10 ft/s) are considered safe in tunnels with no lining. Velocities between about 3 and 6 m/s (10-20 ft/s) usually necessitate concrete linings. For velocities greater than 6 m/s (20 ft/s), the risk of cavitation increases, and special precautions like steel or other types of inner lining must be taken to protect the inside of the structure. Where the water will carry sediments (silt, sand, gravel) the velocity should be kept below 3 m/s (10 ft/s).

(4) A study on friction losses in rock tunnels by Westfall (1989) recommends friction factors (Manning's roughness coefficient, n) for different excavation methods and lining types as follows:

Drill and blast excavation, unlined	$n = 0.038$
Tunnel boring machine excavation, unlined	$n = 0.018$
Lined with precast concrete segments	$n = 0.016$
Lined with cast-in-place concrete	$n = 0.013$
Lined with steel with mortar coat	$n = 0.014$
Lined with steel (diam > 3 m (10 ft))	$n = 0.013$
Lined with steel (diam < 3 m (10 ft))	$n = 0.012$

(5) Factors that can adversely affect friction include overbreak and rock fallout in unlined tunnels, misalignment of precast segments and concrete forms, sediment, and age. Westfall (1989) emphasizes the value of presenting several tunnel diameter and lining alternatives in final contract documents. Huval (1969) presents a method for computing an equivalent roughness for unlined rock tunnels that is employed for different tunnel stretches in an example by Sanchez-Trejo (1985). Figure 6-1 shows the basic



$$\text{Overbreak} = \frac{K}{2} = \frac{D_m - D_n}{2}$$

$$\text{Relative Roughness} = f_m = \left(\frac{1}{2 \log (D_m/K) + 1.14} \right)^2$$

$$\text{Manning's} \quad n = \sqrt{\frac{f_m D_m^{1/3}}{185}}$$

Figure 6-1. Roughness factor calculations for unlined tunnels

equations utilized by this method. Manning's *n* for composite linings of different roughness can be estimated as a weighted average of the friction factors for each surface where length of wetted perimeter of each surface is used for weighting. Figure 6-2 illustrates the variation in friction factor versus flow depth in a shotcrete-lined tunnel with a concrete-paved invert.

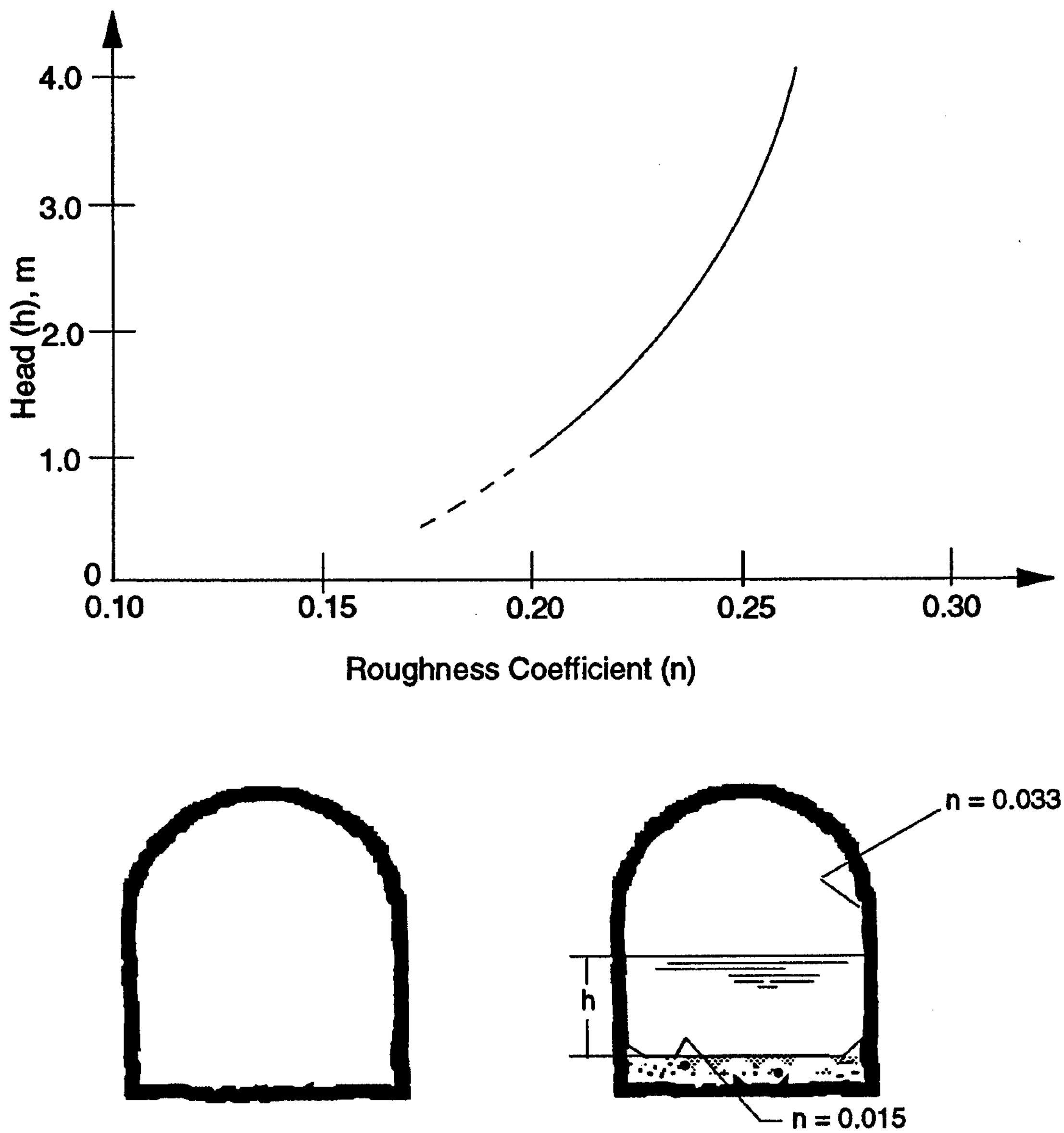
f. Drop shafts for vertical conveyance. Drop shafts are used in water conveyance tunnels to transfer flows from a higher elevation to a lower elevation. Such drop shafts are typically used in flood control and CSO systems. Drop shafts should be designed to dissipate the energy increase associated with the elevation drop; to remove any air that mixes or entrains with the water as it descends; and to minimize hydraulic head losses when the tunnels are surcharged.

(1) *Drop shaft components.* A drop shaft has three essential elements: an inlet structure, a vertical shaft barrel, and a combination energy dissipator and air separation chamber. The inlet structure's function is to provide a smooth transition from horizontal flow to the vertical drop shaft. The drop shaft barrel then transports the water to the lower elevation and in the process dissipates as much energy as possible. At the bottom of the drop shaft, a structure must be provided that will withstand the impact forces, remove any entrained air, and convey the water to the tunnel.

(2) *Basic consideration in drop shaft design.* Several factors must be considered in the design of drop shafts. These factors are variable discharge, impacts on the drop shaft floor, removal of entrained air, and head loss associated with the drop shaft. The selection of an appropriate drop shaft for a particular use involves determining which of these factors are most important. When the difference in elevation between the upper level flows and the tunnel is small, impacts on the drop shaft floor may be alleviated with a simple plunge pool. As the difference in elevation increases, removal of entrained air is necessary and floor impact becomes more severe. In cases where the tunnel hydraulic gradient can rise all the way up to the hydraulic gradient for the upper level flows, head loss also becomes a critical factor.

(3) *Variable discharge.* A drop shaft may be operated for steady-state flows, only during storm discharge periods, or as a combination of the two. The flow variability of a drop shaft has a considerable influence on the design. For instance, for steady-state flow the water surface elevation in the tunnel may be below the base of the drop shaft. In that case, a plunge pool is required at the drop shaft floor to dissipate energy. A shaft that handles only storm flows will not normally require a plunge pool because the water surface in the tunnel will submerge the drop shaft base and cushion the impacts.

(4) *Impact on the drop shaft floor.* The impact of the water on the floor of the drop shaft can be high, and steps should be taken to minimize it. This is accomplished by forcing a hydraulic jump within the shaft, by increasing the energy dissipation due to wall friction as the water descends, by entraining sufficient air to cushion the impact, or by providing a plunge pool at the bottom of the shaft. The plunge pool may be formed by a depressed sump or by the use of a weir located in the chamber at the base of the shaft and downstream of the shaft barrel. The required depth of the plunge pool can be determined by the use of the Dyas formula:



$D_m = 5.275\text{m}$; $D_n = 4.934\text{m}$

$K = 0.341$; $f_m = 0.082$ $n = 0.033$

Figure 6-2. Friction factors for composite lined tunnel (see Figure 6-1 for definition of symbols)

$$Depth = 0.5h^{1/2}d_c^{1/3} \tag{6-1}$$

where

h = height of drop, ft

d_c = critical depth in inlet, ft

(5) *Removal of entrained air.* As the water falls through the drop shaft, it entrains, or mixes, with air. There are several advantages and disadvantages associated with air entrainment. The advantages are as follows:

- Presence of air minimizes the possibility of subatmospheric pressures and thus negates the harmful effects of cavitation.

- Impact of the falling water on the drop shaft floor is reduced by the cushioning effect of the air entrained in the water.

Disadvantages of air entrainment are as follows:

- Flow volume is bulked up and requires a larger drop shaft.

- In order to prevent the formation of damaging high-pressure air buildups, entrained air must be removed before entering the tunnel.

(6) *Head loss associated with the drop shaft.* Under certain conditions the tunnel hydraulic gradient may rise to levels equal to those of the upper level inflow. In these circumstances, the head losses become important because a large head loss may cause severe flooding in the upper level flow delivery system. For example, if this upper level delivery system is a sewer, large drop shaft head losses will result in flow backups into streets and/or basements.

(7) *Types of drop shafts.* Various types of drop shafts have been designed and constructed based on hydraulic laboratory model studies. Drop shafts as deep as 105 m (350 ft) have been constructed. The smaller structures, normally used for drops of less than 21 m (70 ft), are divided into several categories. These categories are drop manholes, vortex, morning glory, subatmospheric, and direct drop, air entraining.

(8) Drop manholes. Drop manholes are generally used in local sewer systems to transfer flows from a higher sewer to a lower sewer. These drop shafts are designed to minimize turbulence, which can release odorous gases and damage the shaft. A typical design has a personnel access upstream of the shaft that allows maintenance personnel to enter the lower sewer without climbing down the wet shaft.

(9) Vortex drop shafts. Flow enters the vortex-flow drop shaft tangentially and remains in contact with the drop shaft wall, forming a central air core as it descends. Since the flows through the inlet are spun against the shaft wall, the entry conditions are relatively smooth. Vortex drop shafts are effective for a wide range of discharges. The air core helps to evacuate the entrained air and to provide near atmospheric pressure throughout the shaft, so as to prevent any cavitation. Vortex drop shafts generally entrain less air than other types of drop shafts for two reasons. First, the flows are highly stable due to the entry conditions. Second, a reverse flow of air occurs in the core of the vortex, which causes much of the air entrained in the flow to be released and recirculated in the zone above the hydraulic grade line. Below the hydraulic grade line, the helical flow has a pressure gradient, which forces bubbles to move toward the center of the drop shaft where they are able to rise against the relatively slower moving water. Therefore, most air entrained by the flow is allowed to dissipate before it enters the tunnel. As the flows are spun against the walls of the drop shaft, significant energy is dissipated before the flow reaches the floor of the drop shaft. The dissipation is a consequence of the wall friction as the flows spiral down at high velocity. The remainder of the energy is dissipated in the air separation chamber by either a plunge pool or by the formation of a hydraulic jump. Several inlet configurations have been adopted to create a vortex flow down a drop shaft (see Figure 6-3). Based on various model studies, a vortex drop shaft is highly efficient when the turned gradient does not approach the level of the upper incoming flow. It is a good energy dissipator and has a high air removal rate.

(10) *Morning glory drop shafts.* Morning glory drop shafts employ a circular crested inlet structure. They are often used as outflows for reservoirs. Model studies have determined that the flow characteristics are controlled by three conditions: weir control, orifice control, and differential head control. The capacity of the morning glory drop shaft is limited by the size of the circular crest. No cavitation is expected in this type of drop shaft. Induced head losses could occur if the circular crest is inadequately designed. The U.S. Bureau of Reclamation recommends that the outlet tunnel be designed to flow 75 percent full to eliminate instability problems.

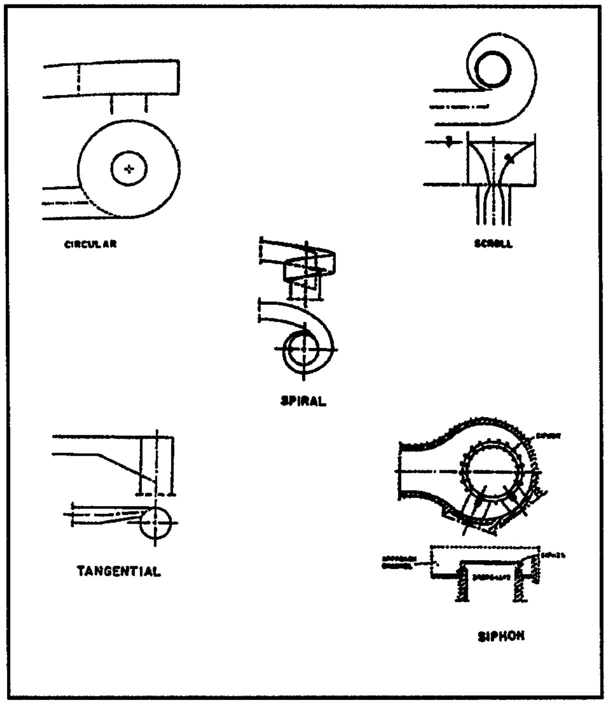


Figure 6-3. Five types of inlets for vortex-flow drop structures

(11) *Direct drop air entraining drop shafts.*

(a) Flow enters these drop shafts radially and descends through the shaft. The shaft diameter is designed to flow full with air entrained in the water, bulking it up enough to fill the drop shaft. The air entrained also provides a cushion for the water, reducing the floor impact. A large separation chamber is used at the base of the shaft and an air vent is necessary to allow the air to vent before entering the tunnel. This type of structure is very effective in dissipating energy and removing entrained air.

(b) Two types of direct-drop air entraining drop shafts are discussed below. The first of these consists of a sump chamber with a sloping top, as shown in Figure 6-4. The air vent is located inside of the drop shaft downcomer barrel, separated by a vertical slotted wall. The slots in the wall allow air to be recirculated into the falling water in the drop shaft resulting in the reduction of large air slugs and providing a more homogeneous mixture of air and water.

(c) At the bottom of the shaft is the sloped-roof air separation chamber. As the air is released from the mixture, it follows the sloping wall of the air collector back up to the air vent side of the vertical shaft and rises to the surface, some of it being recirculated through the slots into the drop shaft. If the drop shaft is to be used for steady-state flows, a plunge pool is built directly beneath the shaft barrel to dissipate the energy.

(d) This structure requires a rather large air separation chamber. For larger drop shafts, this requires a high chamber roof. During the design of the TARP system (Chicago) in rock, it was determined that this type of shaft was economical up to shaft diameters of 2.7 m (9 ft) with a maximum discharge capacity of 17 m^3/s (600 cfs).

(e) Another drop shaft design is suitable for drop shafts larger than 2.7 m (9 ft) in diameter. This drop shaft shown in Figure 6-5 has a separate shaft for the air vent downstream from the downcomer and connected to the downcomer above the crown of the incoming pipe. The air separation chamber has a horizontal roof. The air vent recycles air into the downcomer. This design can be used in much larger drop shafts, up to 6 m (20 ft) in diameter with a maximum discharge capacity of 127 m^3/s (4,500 cfs).

(f) Both structures handle a wide range of discharges and have head losses only one-fifth of those for vortex type shafts. These shafts are the only commonly used drop shafts that can adequately handle variable discharges, impacts on drop shaft floors, remove entrained air, and have minimum head losses to prevent backflow problems when tunnel gradients reach the levels of incoming flows.

(g) The large dimensions of both of these types of drop shafts, particularly the air separation chambers, necessitate mining a major chamber in rock with attendant rock reinforcement and lining. Larger sized versions of these drop shafts can be overexcavated and used as construction shafts.

g. Air removal. High-velocity streams of water may entrap and contain large quantities of air. Air entrainment causes the flow to be a heterogeneous mixture that varies in bulk density throughout the flow cross section and exhibits pulsating density variations.

(1) *Potential problems.*

(a) The engineer should eliminate the harmful effects brought about by the formation of high-energy hydraulic jumps within the tunnel; transient phenomena induced by rapid filling of the downstream end of a tunnel without

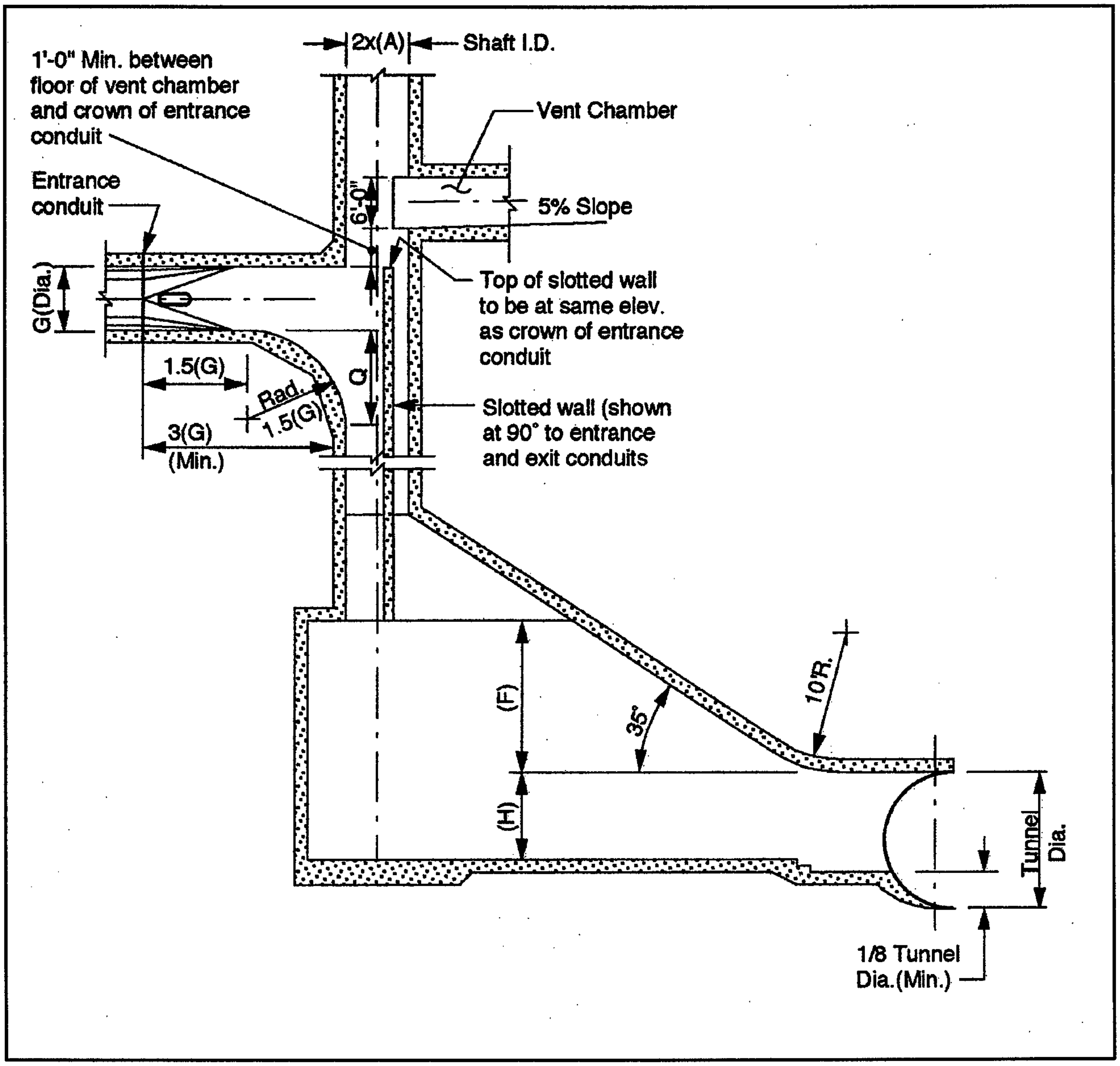


Figure 6-4. Direct-drop air entraining drop shaft

provisions for adequate surge shafts; the formation of air traps within the tunnel system; the introduction of entrained air into the tunnel from drop shafts; and the formation of vortices, which may enter the tunnel through shafts. In addition, the design should provide for the easy egress of air from a tunnel while it fills with water. Improper design can lead to one or more of the following phenomena, which may lead to structural damage:

- Blowbacks—high-pressure releases of air and water in the opposite direction of the flow.
- Blowouts—high-pressure releases of air and water in the same direction of the flow.
- Geysering—air/water venting above the ground surface through shafts located at any point along the tunnel.
- Transient and surging flows causing rapid dynamic instability and possible tunnel collapse.

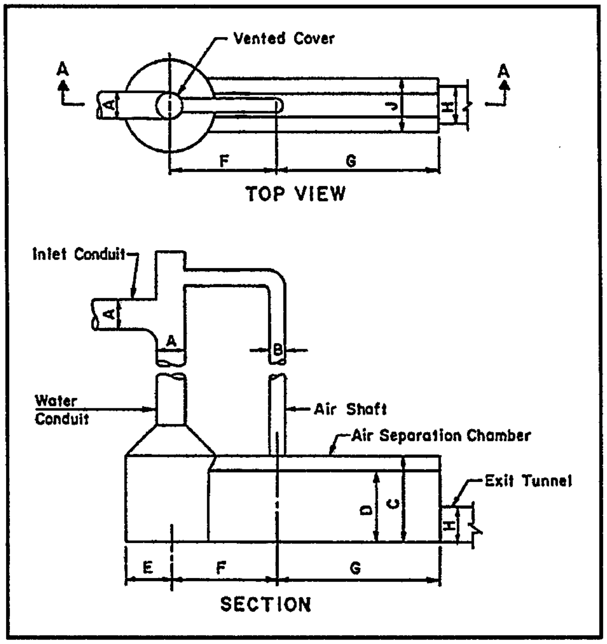


Figure 6-5. Direct-drop air entraining drop shaft with separate air vent

(b) As long as the depth downstream of a hydraulic jump does not reach the tunnel crown, jumps within tunnels are not a severe problem. When the downstream depth seals against the roof of the tunnel, the shock effects of air trapped downstream of the jump can create violent impacts and associated damage. High-energy hydraulic jumps have caused both blowouts and blowbacks. These rapidly escaping air pockets result in water rushing in to fill the voids, creating loud noises and pressure waves, which have resulted in stripping the lining from tunnels and shafts, partial tunnel collapse, and severe erosion.

(c) Even without the formation of hydraulic jumps, blowbacks, blowouts, and geysering, dynamic instability due to transients can take place whenever the downstream end of a tunnel is filling rapidly while air trapped within the system cannot escape at a reasonable rate. When the pressurization surge reaches an upstream end of the tunnel during the filling process, water will rise rapidly in shafts near the upstream end. Water levels in other shafts will also rise as the surge reflected by the upstream end travels downstream.

(d) In pressure tunnel flows, an air void can form at a bend connecting a vertical shaft to a horizontal tunnel. A sudden reduction in the flow rate can cause this void to vent back up the shaft and cause geysering.

(e) Inlet No. 2 of the Oroville Dam Diversion Tunnels experienced the development of vortex. The vortex grew in size and strength as the reservoir filled during the December 1964 flood. After the flood, the tunnel was dewatered and inspected throughout its entire length. Although the observed damage was relatively minor, it did consist of many rough scoured surfaces throughout the entire tunnel length.

(2) *Solutions*. The above-mentioned problems can be prevented by proper precautions during design. The following steps should be taken:

(a) Check the tunnel slopes for the development of supercritical flow and calculate whether a hydraulic jump can occur for any conceivable discharge. A hydraulic jump may not occur during the maximum design discharge but can occur for some lesser discharges. The tunnel slopes should be reduced if the check shows the potential for a hydraulic jump.

(b) Provide surge shafts of diameters at least equal to the diameter of the tunnel at both upstream and downstream ends of the tunnel. A transient analysis should be made during the design phase to determine how high these surge shafts should be.

(c) Whenever branch tunnels or drop shaft exit conduits meet another tunnel and whenever a tunnel changes diameter, always match tunnel crowns rather than inverts, to prevent the formation of air pockets.

(d) Prevent entrained air from entering the tunnel from drop shafts.

(e) Provide a splitter wall to suppress the development of vortices in the inlet to tunnels whenever it is apparent that strong vortex development may occur.

(f) Provide some form of inlet control to regulate or completely shut off all flows into each inlet tributary to the tunnel. This may usually be accomplished by the use of remotely controlled gates at each shaft inlet.

h. Control of infiltration and exfiltration.

(1) The phenomena of infiltration and exfiltration are of critical importance to water conveyance tunnels. Infiltration during construction should be reduced to acceptable levels in all types of tunnels. Significant infiltration after a water conveyance tunnel is completed is unacceptable. Inflows can cause loss of ground into the tunnel and result in surface settlements and damage to neighboring structures. The inflows may cause the adjacent groundwater table to be seriously lowered with resulting adverse impacts on water supply, trees, and vegetation. In flood control tunnels, groundwater infiltration can reduce the carrying capacity available to handle peak flows. Infiltration in water supply tunnels may lead to pollution of the supply. In sewer tunnels, infiltration contributes to increased water reclamation and pumping costs.

(2) Exfiltration from water conveyance tunnels also has potential for undesirable effects. In flood control and sewer tunnels, exfiltration may cause pollution of the adjacent groundwater. Exfiltration from water supply and power tunnels can result in serious reductions in available drinking water and energy supplies as well as revenue loss.

(3) The extent to which infiltration and exfiltration should be reduced must be determined before the design of the tunnel commences. It may be appropriate to apply different standards of water tightness to different sections of the tunnel. It is common practice to specify in the contract documents permissible inflows both during and after the construction of water conveyance tunnels.

i. Lake taps and connection to live tunnels. Connecting a new water conveyance tunnel to an existing high-pressure water tunnel or tapping a lake or reservoir is a task that requires careful advance planning. Obviously such connections are best made in the dry, but in certain cases this is not economically feasible. The following discussion highlights some alternatives.

(1) *Cofferdam.* For tunnels that are to connect to a relatively shallow lake, a ring cofferdam can be constructed from tunnel level below the bottom of the lake to an appropriate elevation above the water surface. The enclosed area can then be dewatered in order to make the connection between the lake and the future shaft and tunnel in the dry.

(2) *In-line tunnel diversion.* To connect a new tunnel to a live high-pressure tunnel, an in-line diversion pipe or series of pipes can be installed within the existing tunnel after it has been temporarily dewatered. A flow cutoff must then be installed around the in-line diversion pipes on both sides of the proposed connection to prevent water from flowing along the backs of the pipes into the connection. With the in-line diversion in place, the new tunnel connection can be made in the dry while the existing tunnel is fully pressurized. When the connection is completed, the existing tunnel may be dewatered again and the diversion pipes and cutoffs removed and the project completed.

(3) *Open-piercing method, lake taps.*

(a) The method is restricted to the construction of a connection in rock. In this method, the new tunnel is advanced as close to the existing high-pressure source as possible, leaving a rock plug in place above the tunnel crown. The tunnel near the connection should be constructed such that, when filled with water, a compressed air cushion will be created below the plug. This air cushion should be maintained until the final connecting blast is made. A rock trap is provided in the invert of the new tunnel below the plug. A shaft from ground surface to the new tunnel invert is also required as close as possible to the connection. A gate is provided on the side of this shaft furthest from the rock plug to seal any water from entering the tunnel beyond the shaft-rock plug section. The rock plug is then drilled and prepared for blasting to make the final connection. Next, the gate is closed and the tunnel (on the rock plug side of the shaft) and the shaft are filled with water to a depth slightly below the water level in the live tunnel or lake to be tapped. At this point, the air cushion below the plug should be checked for adequacy by remote monitoring and additional air pumped in if necessary. The charge is then detonated and the air cushion below the plug interrupts the water column to dampen the pressure shock and prevent damage to the new tunnel. Since the water pressure at the time of the blast is less inside the newly constructed tunnel, most of the rock blasted in the connection will collect in the rock trap. In this procedure, the final connection is left unlined.

(b) There are several other methods to execute lake taps. In 1988, the Alaska District employed the "dry method" for a lake tap for the Snettisham project near Juneau, Alaska. The final plug was about 3.3 by 3.3 by 3.6 m (11 by 11 by 12 ft) and blasted using a double burn-hole cut pattern. A buffer was made of a large plug of ice. Two rock traps were employed.

(c) The design and construction of lake taps and other high-pressure taps must be carried out with the help of specialists experienced in this type of work.

j. Other requirements. The hydraulic requirements of underground structures are of primary importance to design and construction. Other secondary considerations are listed below.

(1) *Construction tolerances.* With open-channel flow, tunnel grade elevation must be established with some precision to maintain the hydraulic properties of the facility. Accurate grade also provides better drainage during construction and avoids accumulation of water in depressions during construction. Grade tolerance for the finished tunnel is usually set at ±13 mm (0.5 in.) for relatively short tunnels, ±25 mm (1.0 in.) for large tunnels. A greater tolerance is given, for constructibility reasons, to tunnels lined with one-pass concrete segments. The centerline tolerance for the finished cast-in-place tunnel is often set at ±25 mm (1.0 in.). However, this tolerance is often irrelevant for functional purposes, and a much greater horizontal tolerance, up to ±150 mm (6 in.) or more can usually be accepted. For a cast-in-place lining, the tolerance on the inside diameter can be set at 0.5 percent, provided the lining thickness is not less than designated. For a precast segmental one-pass lining, a maximum out-of-roundness of 0.5 percent is usually acceptable. Surface irregularities should be kept below 6 mm (0.25 in.).

(2) *Unlined sections may need rock traps.* If a tunnel or shaft is unlined and may collect small pieces of rock or debris, traps are recommended to collect the debris so that it has a minimal effect on flow area, velocities, and friction losses, and so that it will not enter turbines or valves.

6-3. Modes of Failure of Tunnels and Shafts

It is convenient to distinguish between modes of failure that occur during construction and those that occur sometime during the operating life of the structure. Some failure mechanisms observed during construction may be present throughout the operating life if not properly controlled. Some construction failure modes were discussed in the earlier subsection on tunneling hazards (flooding, gases); others more related to the mechanics and chemistry of rock masses are discussed in this subsection. This discussion is not exhaustive because combinations of natural forces and the effects of construction can lead to events that cannot readily be categorized. Nonetheless, an understanding of the forces of nature working in a tunnel environment is helpful in preparing for design work. Failures of tunnels and shafts range from collapse or complete inundation with water and silt to merely disfiguring cracks. They all have underlying causes, and if these causes are understood, the potential exists to discover them ahead of time and prevent or prepare for them. The first set of examples of failure modes are encountered primarily during construction, but some of them may apply to finished tunnels left unlined or with insufficient ground support. The second series of examples apply to finished, lined tunnels. Failure of environmental nature, such as detrimental groundwater drawdown or damage due to settlements are discussed in Section 5-14.

a. Tunnel and shaft failure modes during construction.

(1) *Failures controlled by discontinuities.*

(a) Rock masses are usually full of discontinuities, bedding planes, fractures and joints, or larger discontinuities, faults, or shears that may form zones of weakness. These are planes of weakness where the rock mass may separate or shear during excavation. Whether or not they will separate or shear and cause a rock fall into the tunnel is largely a matter of geometry, and of the tensile and shear strength of the discontinuity.

(b) The tensile strength across a bedding plane is often poor or nonexistent. The shear strength, however, can be close to that of the adjoining materials, depending on the normal stress across the plane, as well as joint roughness and other surface characteristics. Because the excavation of a tunnel results in a general unloading of the tunnel environment, the shear strength of a bedding plane is often greatly reduced, depending on the orientation of the bedding plane relative to the opening. Therefore, bedding planes often participate in forming blocks of rock that can fall from tunnel roof, wall, or face.

(c) Shaley beds in a sandstone or limestone formation may appear to be sound at first exposure, but the unloading due to excavation combined with access to air and water can soften and cause slaking in such beds in hours or days such that they lose most of their tensile and shear strength and participate in the formation of rock falls. It is common in such bedded formations to experience rock falls days after excavation.

(d) Joints and fractures have no tensile strength, unless they have been healed by secondary deposition of minerals. The shear strength of a joint depends on a number of factors: Width, infilling (if any), local roughness, waviness on a larger scale, the strength of the joint wall (affected by weathering), and the presence of water.

(e) One discontinuity across or along the tunnel cannot form a block that will fall from the roof, wall, or face.

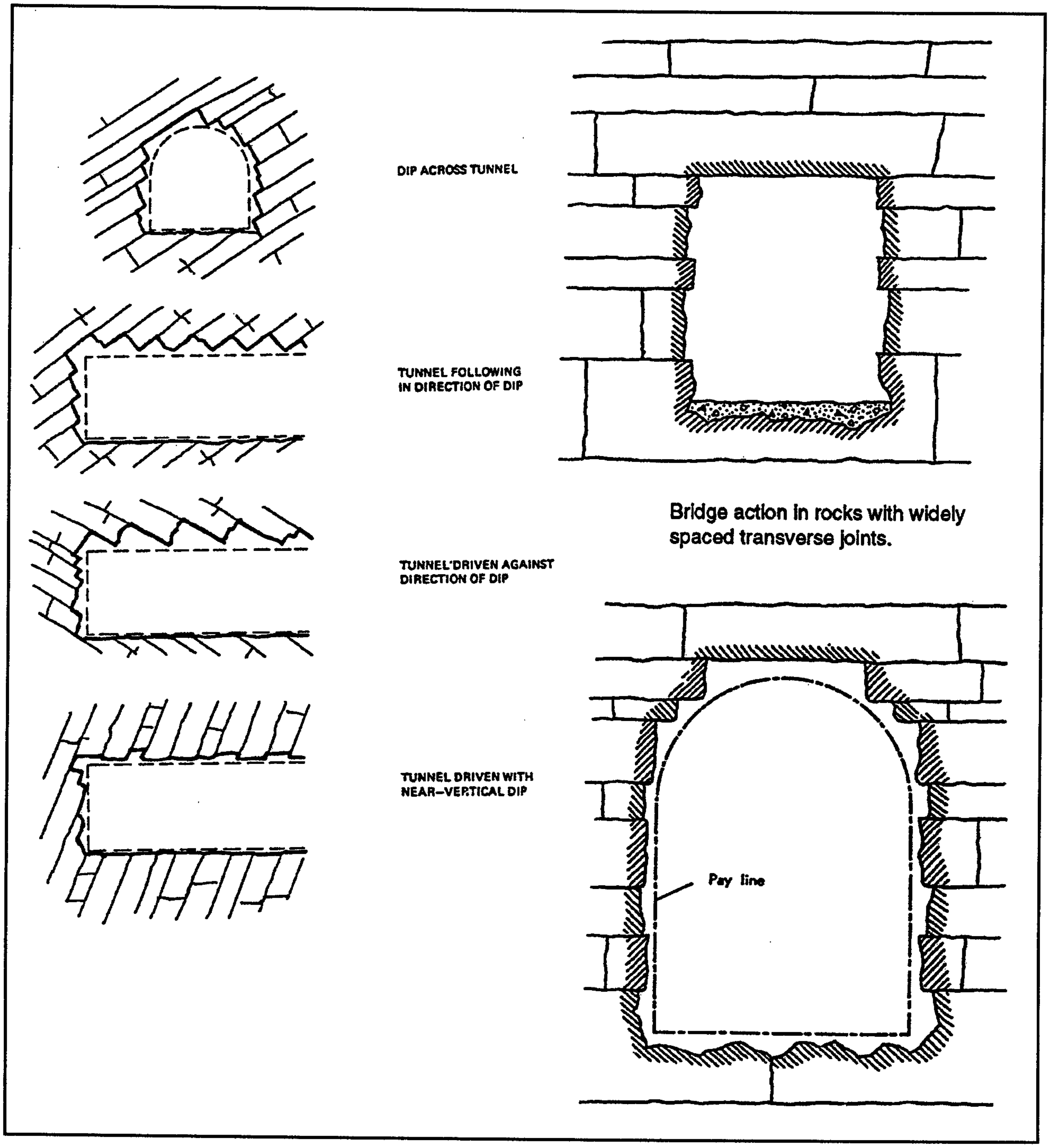


Figure 6-6. Examples of discontinuities (in part after Proctor and White 1946) (Continued)

It usually takes three intersecting discontinuities to form a loose block. However, gravity can help cause a cantilevered block to fail by bending or tension, and stress concentrations around the opening can result in other unfavorable fractures through intact rock, causing rock falls even with only two (sets of) discontinuities. Figure 6-6 shows several examples of how fractures and bedding planes can affect tunnel stability.

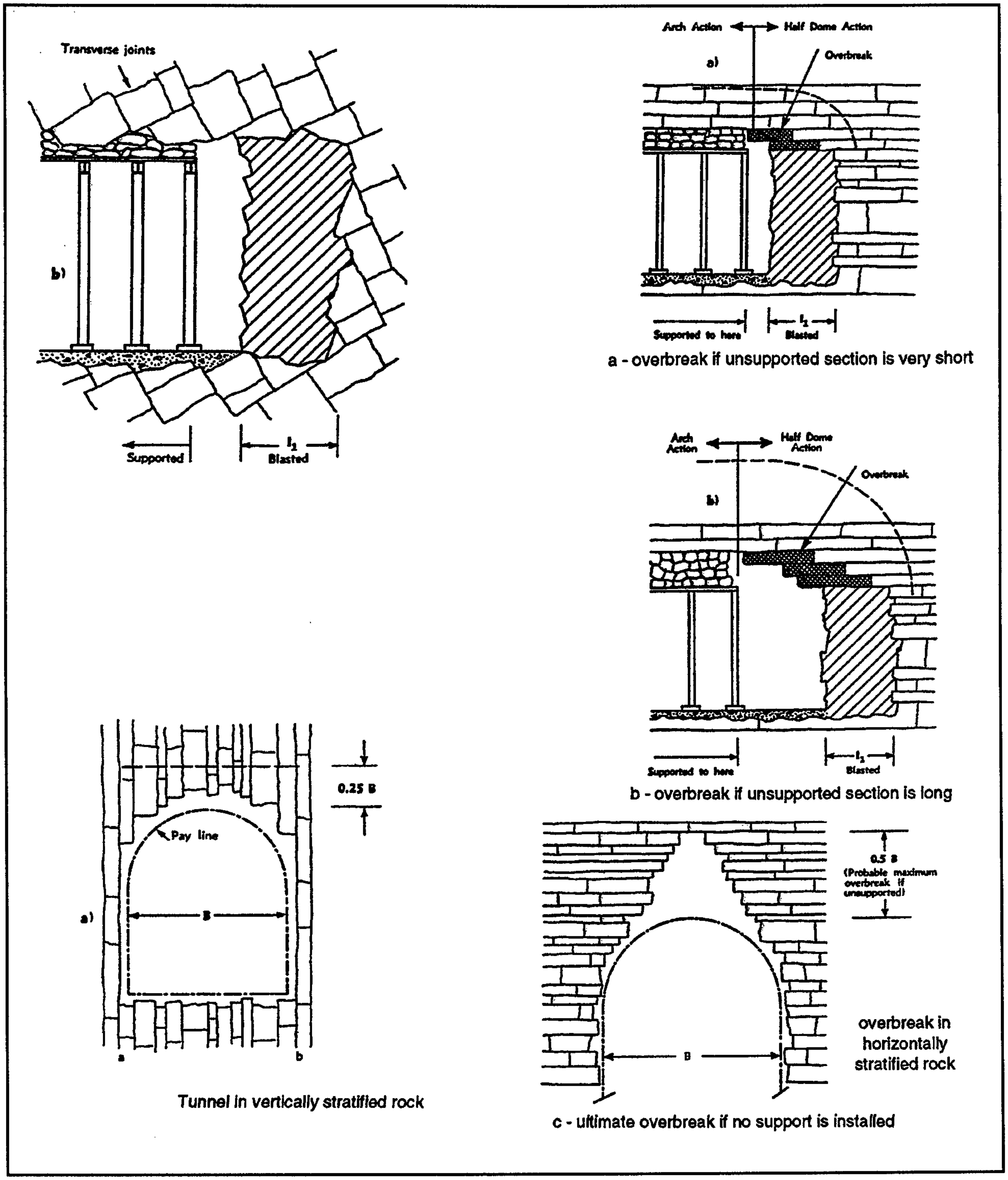


Figure 6-6. (Concluded)

(f) If orientations and locations of discontinuities were known before tunnel construction, stability of blocks could in theory be predicted using graphic techniques or block theory (Goodman and Shi 1985). For a long tunnel, this is not feasible. If only orientations are known, with an idea of the spacing or frequency of the discontinuities, then an assessment can be made of the probability or frequency of potential rock falls. On this basis, a rational determination can be made of the need for ground reinforcement (e.g., in the form of systematic or spot rock bolts or dowels) and the most effective orientation of such ground support.

(g) When tunnels are excavated by blasting, excess blasting energy at the perimeter will cause damage to the surrounding rock. This damage manifests itself as a loosening and weakening of the rock mass. With poor, uncontrolled blasting practices, the zone of damage can reach a distance of one to several meters. Joints and other planes of weakness may open temporarily or permanently due to the pressure of escaping gases or the dynamic, mechanical effect of the blast, thus eliminating any tensile strength that might have been available and reducing the shear strength. The blast will also create new fractures. Combined with the stress reduction due to the excavation of the opening, these effects greatly increase the opportunity for rock falls. An opening that would otherwise have been stable could require considerable ground support due to effects of poor blasting.

(h) Jointed and otherwise flawed rocks can be classified in many ways. One method of classification is described in Section 4-4, Terzaghi's classification of rock conditions for tunneling purposes. Additional comments are presented below.

(i) For purposes of underground design, intact rock may be described as rock in which discontinuities are spaced such that, on the average only about three to five discontinuities intersect the tunnel. Examples are massive igneous rocks, marbles, or quartzites with widely spaced joints, and sedimentary rocks that have been left largely unaffected by tectonics, dolomites, limestones, shales, and sandstones sometimes qualify.

(2) *Stratified rocks*. Stratified rocks are sedimentary or metamorphosed rocks with distinctive layering, where bedding planes are potential planes of weakness. Schistose rocks are typically metamorphosed rocks with layers or planes of weakness that are often greatly contorted.

(3) *Moderately and highly jointed rocks*. These rocks display few, if any, bedding plane weaknesses, but joints crossing the tunnel may number 10 to 100. Joints often come in patterns, with one to three sets of joints, each set containing mostly subparallel joints but the joint sets intersecting each other at angles. These kinds of rock are often called blocky or very blocky.

(4) *Interlocking rocks*. Interlocking, jointed rock masses can be moderately or highly jointed, but the joints are tight and contorted such that their inherent shear strength is high. Examples are some basalts, welded tuffs and rhyolites, and other rock masses where the jointing is largely the result of tension fracture from cooling soon after original deposition. Interlocking, jointed rock is often stable with a minimum of ground support.

(5) *Blocky and seamy rocks*. Blocky and seamy rocks combine jointing with weak bedding planes or schistosity. In sedimentary rocks, one or more joint sets are often seen at roughly right angles to the bedding planes.

(6) *Shattered or crushed rock*. This consists of mostly chemically intact fragments of rock, which may or may not be interlocking; the fractures are sometimes partly rehealed. Fault zones often contain rock that has been completely sheared into a silty or clayey material of low-strength, fault gouge. Such gouge is often responsible for squeezing conditions. The Karawanken case history (see Box 6-1) is a dramatic example of tunnel collapse in a fault zone. Missing in these descriptions is an indication of the degree of alteration and weathering. As earlier noted, weathering can have a profound effect not only on the strength of the joints but also on the intact rock strength. Recommendations for ground support based on these descriptions, intended for the design of steel sets, were formulated originally by Terzaghi. These recommendations are found in Chapter 7.

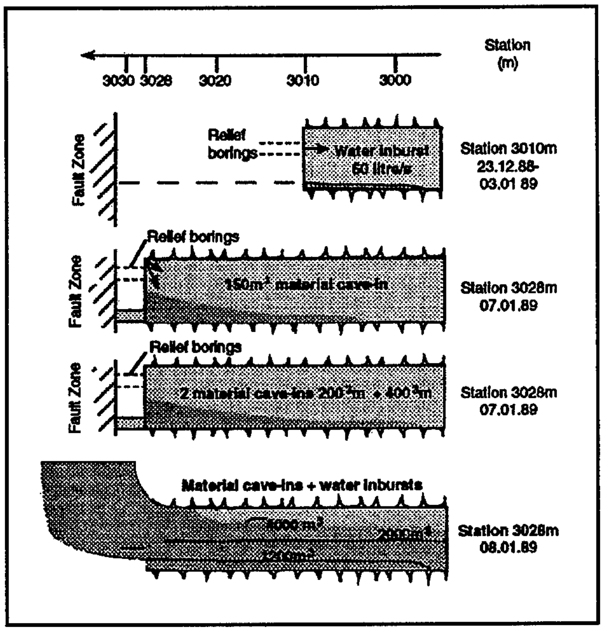
(7) *Rock failures affected by stresses*.

(a) Before excavation of an underground opening, the stresses in the rock mass are in a state of equilibrium. Excavation will reduce or eliminate the stress normal to the wall of the opening, while at the same time increase the stresses in the tangential direction through stress concentration, an effect similar to the development of stress concentrations around holes in plates. The effect of the increase in tangential stress depends on the strength of the rock, its ductility, and the stress distribution in the surrounding rock.

(b) If the rock is overstressed, it will yield or fail. A plastic, ductile rock (e.g., shale), behaving similar to a clay, may yield without losing coherence while the yield zone sheds load to deeper, unyielded rock. A fractured

Box 6-1. Case History: Karawanken Tunnel Collapse

The Karawanken Motorway Tunnel between Austria and Slovenia was built in 1987-91. The tunnel is 7.6 km long, with a 90-m^2 cross section and a maximum cover of nearly 1 km. This tunnel experienced a very large collapse during construction in 1988.



The tunnel traverses a variety of sedimentary rocks, ranging from dolomites and limestones to marls, clay shales, and conglomerates. The strata are severely folded and cut by a number of fault zones.

Excavation was by blasting methods, pulling 0.8-3.5 m with each blast, with a crown heading followed by bench removal at 80-150 m from the crown face. In some poor areas, two benches were employed. Ground support consisted of shotcrete varying in thickness from 50 to 250 mm, supplemented with rock bolts and steel mats as well as steel arches, based on a rock classification system. Where squeezing ground was encountered, open slots were left in the shotcrete application at the crown to permit displacements and rock relaxation. The construction procedure relied upon stabilization by pre-drainage, using horizontal bore holes from the face of the tunnel.

The collapse occurred at Sta. 3028, close by the Slovenia-Austria border and near the greatest amount of cover. Here is an abbreviated version of the series of events (see figure).

1. Dec. 23, 1988: At Sta. 3010, two exploratory borings encounter water, and large quantities of water and sand are released.
2. Dec. 27, 28: Five relief holes carry 60 l/s of water but soon collapse.
3. Jan. 3, 1989: Recommence driving after break; 1.2 m advance per round, relief drainage.
4. Jan. 7: At Sta. 3028 a crown borehole releases a water inburst, carrying 150 m^3 of material. Later a 500-mm drainage hole is drilled. This hole caves and delivers 200 m^3 of material, followed later by an additional 400 m^3 of material.
5. Jan. 8: The caved 500-mm hole is reopened by a small explosives charge. This is followed by more water and material inburst. Later on the same day the face collapses suddenly, releasing about 4,000 m^3 of water and material.

The causes of the failure were diagnosed to be a combination of at least the following factors:

1. Wide fault zone consists of crushed dolomite with sand and clay joint infill.
2. Removal of sand and clay material from the joint fillings result in loosening of the rock mass and loss of confining pressure.
3. Strength of the rock mass is reduced due to water softening, high water pressures (up to 35 bar), and reduced confining pressure.
4. Supporting pressure at the face was removed by excavation.
5. A contributing factor was the lengthy New Years break, during which water and fines were permitted to drain from the face.

Remedial measures consisted of placing a concrete bulkhead in the tunnel, constructing a bypass, placing a 5-m-thick ring of grout by injection, and careful remining.

If the potential seriousness had been recognized in time, the failure might have been prevented by grout injection into the entire width of the fault zone to make the zone impermeable and stable.

Reference: Maidl and Handke (1993).

rock, held in place by a nominal support of dowels or shotcrete, may yield with small displacements along fractures, perhaps with some fresh fractures, again shedding load to more distant, stronger rock. On the other hand, if the fractured rock is not held by a nominal force, pieces may tend to loosen, resulting in a stress-controlled raveling situation. A stronger, brittle rock will fracture and spall. A very strong rock can store up a great deal of elastic energy before it breaks, resulting, then, in occasionally violent rock bursts.

(c) The strength of intact rock as well as that of a fractured rock mass usually depends on the confining pressure. Just like a frictional soil material, the strength increases with the confining pressure or the minimum principal stress. Around an opening, the minimum principal stress is the pressure in the radial direction. Zero at the wall of an unlined opening, it increases rapidly when the wall curves but not when it is straight; the sharper the curve, the more rapid the increase in confining pressure.

(d) As it turns out, the highest stress concentrations are usually at the sharpest curves, such as the lower corners of a horseshoe-shaped opening, but here the confining pressure increases so rapidly with distance that a little local yielding tends to stop the process of failure. On the other hand, low-stress concentrations are often found around flat surfaces, such as flat roofs or floors (inverts). Here the stress gradients are small and stress fracture, when it occurs, can be very extensive. This is exacerbated in a rock formation that is horizontally stratified with little bond between the strata; here such stress conditions can lead to buckling.

(e) On occasion, tangential stresses induced over the crown of a tunnel will help confine blocks of rock that might have loosened in the absence of such a confining stress.

(f) Stress effects, then, depend on (at least) the following factors:

- Induced stresses, which depend on in situ stresses and opening shape, and the distance from the advancing face of the excavation.
- Rock strength; the intact rock strength can be measured; the operating parameter is the ratio between induced stress and rock strength, or if the induced stress is undetermined, the in situ overburden stress to rock strength ratio.
- Rock modulus and ductility, also measurable.
- Stress gradient, which can be calculated just as the induced stresses.
- Effect of fractures on strength and ductility, not measurable and barely possible to guess.
- Effects of stratification.

(g) Box 6-2 shows a method of assessing modes of failure based on induced stress level, rock strength, and rock quality. Box 6-3 describes various manifestations of stress-induced failure based on rock type and rock strength.

(h) As discussed later in this section, one type of stress-controlled failure is squeezing. This is a slow or rapid encroachment of rock material into the tunnel, without change in water content. In a soil, this would be likened to the squeezing or flow of a soft clay into the face of a shield, when the overburden pressure exceeds about six times the undrained shear strength of the clay. In a rock tunnel, squeezing conditions are often found in fault zones with altered or weathered material of low strength. At great depth where the stresses are high, a low-strength fault-zone material can result in a great deal of squeeze, and loads on a lining can approach the overburden pressure.

(8) *Failure modes affected by mineralogy.*

(a) Some modes of failure in tunnels are largely controlled by properties of the intact material. The concept that the strength of a massive rock affects stress-controlled failures such as spalling, rock bursts, or squeeze has already been discussed. Properties other than the rock strength also can result in failure or unacceptable behavior.

(b) Poorly consolidated shales or marls or shaley and marly layers in a limestone can slake when exposed to air and moisture. This is a phenomenon brought about by the stress relief combined with drying and wetting, and it appears in the tunnel as loosening of flakes or chunks of material, sometimes partly controlled by bedding. As pieces of the rock fall off, more rock gets exposed; slaking with time can result in the loosening and removal of several feet of rock. Slaking is greatly accelerated if water is permitted to enter the latent fractures of the rock and soften the rock. The risk of slaking can be assessed by means of laboratory tests, as discussed in Section 4-4.

(c) Saturated clay-like materials, when unloaded, will often generate negative porewater pressures (suction).

Box 6-2. Assessing Mechnaical Modes of Failure

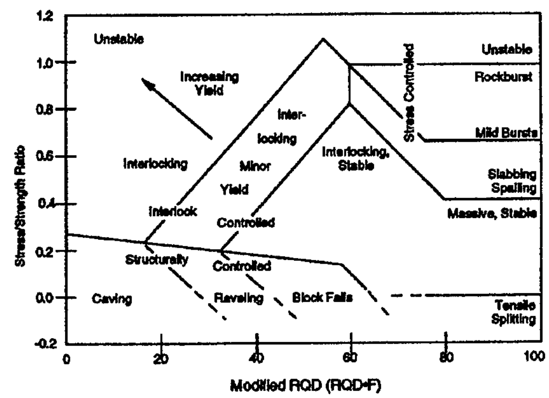
1. Behavior of Strong and Brittle Rock Based on RQD and Induced Stresses

The following method of assessment was developed for nuclear waste repository design (Schmidt 1988) and is applicable to brittle, jointed, interlocking rocks, such as basalt, welded tuff or rhyolite, as well as other massive or jointed rocks, such as quartzite, marble, and most igneous and metamorphic rocks.

The method is based on the premise that massive rocks subjected to high stresses will suffer stress failure, but that flaws in the rock mass will permit relaxation of high stresses, leading to the potential for other modes of failure. The method requires the calculation of the stress/strength ratio, defined as the ratio between maximum tangential stress induced around an opening (calculated by closed solutions or numerical methods) and the unconfined compressive strength of the intact rock. The effect of flaws is assessed using a modified RQD, as follows:

$$\text{Modified RQD} = \text{RQD } F_1F_2F_3F_4F_5,$$

where



F_1 = factor for joint expression on a large scale (waviness), on a small scale (roughness), and continuity. Range 0.9 to 1.0 (1.0 for very wavy, rough, and discontinuous joints)

F_2 = factor for joint aperture and infilling, and joint wall quality. Range 0.92-1.0 (0.92 for soft or weakened joints)

F_3 = factor for joint orientation, favorable, random, unfavorable. Range 0.9-1.0

F_4 = factor for blast damage. Range 0.8-1.0 (1.0 for TBM tunnel, 0.8 for poor blasting)

F_5 = scale factor, function of ratio between opening size and joint spacing. Range 0.85-1.0

Ratio: Opening span/Joint Spacing	<4	4-10	10-30	>30
Factor F_5	1.0	0.96	0.88	0.85

The figure shows the predicted types of ground behavior based on stress/strength ratio and modified RQD. As most such charts, it is conceptually accurate, but the bounds between regions of behavior are imprecise and subject to judgment. For example, a jointed rock mass with joint blocks that are not interlocking (most tectonic joints) would most likely display a larger region of structurally controlled behavior.

These materials will absorb water either from the air in the tunnel or from distant regions in the clay mass, resulting in swelling. If unsupported, the clay mass will encroach on the tunnel profile; if lined, the tendency to swell will be halted but will result in lining pressures. Tertiary clays in Europe have been known to produce lining pressures greater than the overburden pressure. This is possible because these types of overconsolidated clays are usually subjected to in situ horizontal stresses greater than the vertical stresses.

(d) Prediction of swelling pressures in saturated clay or clay-shale has often been attempted using swell tests of the types used to predict swelling of unsaturated clays at the ground surface. However, because the swelling of a saturated clay in the underground is an entirely different phenomenon than the swelling of an unsaturated clay at the surface, these tests are useless for the purpose. Such underground swelling pressures, in theory, can be predicted by soil-structure analysis, but the necessary data to perform these analyses are difficult to obtain. Experience shows that the amount of swell of a clay or clay shale depends on the degree of cementation between clay particles; however, hard and fast general rules have not yet been established.

(e) Unsaturated clays or clay-shales are sometimes found in tunnels. These can be more prone to swelling than are the saturated materials, and standard swell tests performed on unsaturated samples can be useful. When

Box 6-3. Assessing Mechanical Modes of Failure

2. Manifestations of Stress-Controlled Failure

Unconfined Compressive Strength ksi	MPa	Typical Rock Types	Overstressed Behavior For Massive Rock	For Jointed Rock
64	440	dense basalt, quartzite, diabase, gabbro	violent regional, local rock bursts	
32	220	granite, most igneous rocks, gneiss, strong metamorphic marble, slate	breakouts in boreholes lesser rock bursts spalling, popping	combined failures (joints, intact rock)
16	110	hard, dense, sedimentary, welded tuff, dolomite, limestone	spitting, hour-glass pillars	
8	55	schistose rocks		
			flaking	
4	28	phyllite		
2	14	lower density sedimentary, chalk	stress slabbing	
1	7	tuff	slow slabbing	
0.5	3.4	marl, shale	squeezing slaking of poorly cemented shales	swelling accelerated by water access to joints
0.25	1.7	weak clay shale	swelling when cementation destroyed	
0.13	0.8	weathered and altered rock	ravelling of fissured clays	
0.06	0.4	hard clay	yielding of nonfissured clays	

Other effects:

- Stress-induced creep in halite, potash
- Swelling of anhydrite (up to 2 Mpa swell pressure with access to water)
- Dissolution of soluble materials

Note: Approximate lower limit for violent rock bursts: 18-24 ksi (125-165 MPa)

such materials are exposed to water during tunneling or due to leakage from the tunnel after completion, they can generate substantial swelling pressures. Such modes of behavior are accelerated by preexisting fractures (common in such materials) or fractures resulting from excavation and stress redistribution. The Peace River diversion tunnel case history (see Box 6-4) is an illustration of the effect of water on a silty shale.

(f) A common failure in weak, shaley rock, particularly in tunnels with a flat floor (horseshoe-shape) and high in situ horizontal stresses, is excessive floor heave. This type of failure is the result of several factors:

- For most in situ stress conditions, a flat floor results in very low vertical and often high

Box 6-4. Case History: Diversion Tunnel in Soft Shale, Peace River

For the Site 3 hydroelectric project in British Columbia, three diversion tunnels through the left abutment were proposed. Confidence in the behavior of the soft shale was not great, and a test chamber in the shape of a truncated cylinder, 11.1 m wide, 7.5 m high, and 45 m long was excavated. The chamber is at a 107-m depth and connected to the canyon wall through an adit. Most excavation was by roadheader, but part of the adit and part of the chamber were excavated by controlled blasting.

The geologic material is a Cretaceous, horizontally bedded silty shale with about 10 percent smectite, with unconfined compression strength 6 Mpa (900 psi) and modulus 3-4 GPa (440-580 ksi) perpendicular to bedding, 6-8 Gpa (870-1,160 ksi) parallel to bedding. The material is prone to slaking and weathering when exposed. Bedding plane fractures are common, as are steeply dipping relaxation joints parallel to the canyon wall.

Ground support included two layers of fiber shotcrete and tensioned resin dowels spaced 2 m. The chamber was instrumented with convergence gages, multipoint extensometers, and stress cells.

The chamber was successfully excavated and supported, using heading and bench. Shotcrete in the roadheader section was generally sound, with minor shrinkage cracking, but in the blasted section up to 65 percent of the shotcrete was drummy.

After completion, the chamber filled with water, 5.5 m deep, for about 2 years; it was then pumped dry and inspected. Shotcrete in the crown, which remained dry, had remained virtually unchanged and sound. Below the water line, the shotcrete was badly cracked and spalled, and drummy throughout. Two block falls of 100-150 m^3 each had occurred, bounded by clean joints parallel to the tunnel wall. Cores were taken, and shale from the wet zone was found to be soft and fissile. Ground movements in the dry crown were about 0.3 mm, but in the wet zone, ground movements amounted to 50-120 mm.

Conclusion: Shotcrete-shale bond was a problem if the shotcrete was not applied quickly; more so in the blasted than the mechanically excavated parts. Water found its way through cracks and voids in the shotcrete into existing and latent fissures in the shale, where it caused softening and swelling, and resulted in displacement and spalling of shotcrete. The diversion tunnels are to be designed with a circular shape and a cast-in-place concrete lining over the initial shotcrete support.

Reference: Little (1989)

horizontal stresses in the floor, conducive to swelling of the floor material.

- Seepage water finds its way to the floor, causing swelling.

- The floor is subject to construction traffic, which causes softening in the presence of water.

Swelling also occurs when geologic materials such as anhydrite or shales containing anhydrite absorb water.

(9) *Effects of water.*

(a) As discussed earlier, groundwater contributes to modes of behavior such as swelling and slaking. Water can contribute to many other modes of behavior and failure.

(b) Some rocks or minerals are soluble in water. These include, most notably, halite (rock salt) and gypsum. Moving water will carry away salt and gypsum in solution and leave behind voids that can cause increased water flow and accelerating dissolution. Voids can cause surface subsidence or irregular loading and loss of support for tunnel lining and the ground support system. Removal of the gypsum cement in a sandstone by seepage water has caused the failure of at least one major dam (San Franciscito Dam in California, in 1928). When such materials are present, particular attention must be paid to the watertightness of the tunnel.

(c) In the longer term, limestone is also subject to dissolution. In this case, however, the concern is more for the likelihood of encountering voids and caverns than the prolonged effect of dissolution on the tunnel structure.

(d) Flowing water will erode unconsolidated material. Piping phenomena are common in soils, where backward erosion by seepage water can cause failure of dams and excavations as well as cut slopes. In rock masses, joint fillings and crushed fine materials in faults and shear zones are particularly susceptible. In a construction situation, prolonged water flow out of joints and shear zones can cause serious weakening of the rock mass by removal of fines, resulting in loosening and potentially collapse (see

Box 6-1 on the Karawanken Case History). Contributing factors in such situations are the weakening effect of the water on the strength of intact rock and joints, joint fillings, and gouges; the hydrostatic pressure reducing the effective stress across joint surfaces; and the seepage forces of the flowing water.

(e) Inflow into tunnels loaded with silt and sand will cause maintenance problems for dewatering pumps. An open TBM is not greatly affected by water inflow, but a shielded TBM often suffers problems when inflows exceed several tens of liters/second (several hundred gpm), especially when the water brings in fines. Often the mucking system, whether by rail cars or conveyor, is overloaded by the water, and water with fines escapes the system, resulting in deposition of fines at locations where they will be troublesome. As an example, silt deposited in a telescoping shield joint will cause wear in the joint and may destroy waterproofing gaskets. Silt deposited in the invert can seriously hamper placement of invert segments. Excess water can also affect the electrical system and cause corrosion of tunneling machinery, especially if the water is saline or otherwise corrosive.

(f) Inflow into tunnels will tend to drain the rock mass and any overburden. This, in itself, may be unacceptable, especially if existing flora or operating wells are dependent on maintenance of the groundwater table. Lowering the groundwater table can also result in consolidation of unconsolidated materials, especially soft clays, resulting in unacceptable surface settlement.

(g) A particular type of failure mode applies to water tunnels in which the water pressure fluctuates, such as in power tunnels with surges and water hammer effects. If the tunnel is unlined or supported only by rock bolts or dowels, the fluctuations in water pressure can result in water flushing in and out of rock fissures, eventually cleaning out joint fillings. This also happens if there are cracks in a tunnel concrete or shotcrete lining that permit the flushing of joints. More than one power water tunnel has failed by collapse in this way.

(10) *Particular failure modes for shotcrete.*

(a) Before reviewing failure modes for shotcrete ground support, it is useful to recapitulate the various functions and actions of shotcrete support, when applied to a minimum thickness of 50-75 mm:

- Sealing coat to prevent atmospheric deterioration, slaking, drying, wetting, swelling.

- Prevents blocks of rock from falling out by shear and bond strength; prevents smaller fragments from falling and start a raveling sequence.

- By shear, bond, and bending to withstand local forces or forces of limited extent (local blocks, seams subject to squeezing or swelling).

- As a compression arch or ring, to withstand more-or-less uniform loading from squeezing, swelling, or creeping ground.

- Provide some degree of water inflow control.

- In combination with rock bolts or dowels, provide overall stabilization and ground movement control.

(b) Overall, by inhibiting ground motions and supplying a confining pressure for the rock mass, the shotcrete acts to retain and improve the strength of the rock mass and to help in creating a self-supporting ground arch in the rock mass.

(c) Where shotcrete is a part of initial ground support, to be followed by subsequent installation of a final lining (whether by cast-in-place concrete or additional shotcrete), performance requirements are less stringent than when shotcrete is the final support. Shotcrete as initial ground support can be repaired and even replaced as required, and even significant flaws can be tolerated, provided they do not impair the safety of personnel. The principle of controlled deformation of initial shotcrete support is discussed further in the section on the New Austrian Tunneling Method (Section 5-5).

(d) Some failure modes of shotcrete result from imperfections in its application, others from properties and nonuniformities of the rock mass, or the action of formation water. Some examples follow:

- Shear failure resulting from loss of (or lack of) bond between rock and shotcrete, usually initiated by nonuniform loading combined with an incomplete ring of shotcrete.

- Shear failure from local block load or load from a seam of squeezing material.

- Compression failure from excessive external uniform or nonuniform load, sometimes a combined bending and compression failure.

- Fracture due to excess external water pressure, resulting in excessive water inflow, sometimes resulting from plugging of geofabric strips and piping provided for draining the rock mass.

- Shear failure of shotcrete around a rock bolt or dowel plate resulting from excessive displacement (squeeze) of the rock mass.

(e) Loss of rock-shotcrete bond can result from incomplete preparation of a wet, partly deteriorated rock surface or one covered with grime, dust, or mud. Other common flaws are areas with too little or too much aggregate, too high water/cement ratio, imperfect application of admixtures resulting in slow curing, or too thin an application. Application of shotcrete in a location with flowing water can result in washouts or imperfect bonding or curing.

(f) The case history in Box 6-4 shows failure modes of shotcrete exacerbated by fractures in the shotcrete and softening of the rock.

(g) Many potential modes of failure of a shotcrete application are functions of flaws in shotcrete application and local variations in geology and loading, generally not subject to analysis but usually controllable during application. Where the shotcrete forms a structural arch or ring bonded to the surrounding medium and subject to external loads, the shotcrete structure is amenable to analysis.

(11) *Failure modes of rock bolt or dowel installations.*

(a) Rock bolts or dowels can control or reduce displacements, both initially and in the long term, by preventing loosening of the rock mass and increasing the rock mass modulus to hold rock blocks or wedges in place. In a pattern, they act to form a reinforced arch or beam capable of sustaining loads that may be uniform or nonuniform. By preventing loosening of the rock mass and by increasing the rock mass modulus, bolts and dowels control or reduce displacement in the short or long term. Prestressed bolts induce compression in the rock mass, further increasing its strength and carrying capacity and reducing displacements. Bolts and dowels are often supplemented by metal straps, wire fabric, or shotcrete.

(b) Bolt or dowel installations may be considered permanent parts of the underground structure, or they may be temporary and not counted on for permanent support. The installation may be supplemented at any time with additional ground support elements.

(c) Individual dowels or bolts can fail in either shear or tension in the steel, or yield can occur along the bond between grout and rock, or between metal and grout. Sometimes failure occurs due to faulty installation (insufficient grout, grout not properly set, improper anchoring).

(d) A systematic bolt or dowel installation can fail by loosening, raveling, or block fall between individual bolts; this depends on joint spacing relative to bolt spacing and the degree of interlock between rock blocks. If bolts are too short to anchor a large wedge, such a wedge can fall out, bringing down one or several bolts with it.

(e) A systematic bolt or dowel installation forming an arch or a beam can fail due to overstress of the reinforced rock mass. This usually indicates that the bolt length chosen was too short.

(f) In a soft, squeezing ground, bolt face plates can fail by overload in the metal or by punching failure into the rock.

(g) Whether any of these modes of performance have serious consequences depends on the permanency of their installation. Systems installed for temporary purposes only are considered to perform acceptably as long as there is no hazard to personnel and the permanent lining can be installed without problem. The temporary installation is employed to arrest ground movements before permanent lining installation.

(h) When the bolt installation is considered as part of the permanent installation, some of these modes of failure may still be acceptable. Yielding of part of the system (shear, tension, bond) may be acceptable as long as the rock mass is coherent and deformations are under control. However, their value may have to be discounted for the design of the final lining. Any behavior mode that can result in future corrosion, however, usually requires that the element is ignored for final design consideration.

(12) *Particular failure modes for shafts.*

(a) Because shafts are oriented 90° from tunnels, some modes of failure are more or less common than for tunnels. There are several reasons for that. First, since a shaft penetrates the geologic strata in a vertical direction, a shaft is likely to encounter a greater variety of conditions, including overburden and weathered rock. Second, gravity acts on the shaft wall like on a tunnel wall, much less severely than on the crown of a tunnel. Third, methods of shaft construction are generally very different from

methods of tunnel construction, as discussed in Section 5-7. The following are a few examples of shaft failure mechanisms.

(b) Shaft bottom failure is usually caused by water pressures. With an impervious plug above an aquifer at the bottom of the shaft, the plug can fracture or burst if it is too thin and cannot hold the pressure, whether by bending failure or shear along the sides, or some combination. Of course, sinking the shaft and ignoring the aquifer altogether could result in flooding of the shaft, if the permeability in the aquifer is sufficiently great.

(c) Grouting or freezing is often used to control groundwater inflow and the effect of groundwater pressures during shaft construction. It is difficult to ascertain the quality of grouting, and ungrouted zones can be left that would result in excess inflow of water, perhaps carrying solids, when encountered during sinking. A freeze-wall occasionally fails, also resulting in inrush of water, often because flowing groundwater brings caloric energy to the site and thaws the wall.

(d) Another shaft failure mode has nothing to do with rocks or groundwater but with the site arrangement: flooding of the shaft from surface waters. This type of incident is inexcusable; shafts constructed anywhere near a floodplain must be equipped with a collar tall enough to prevent flooding.

(13) *Particular failure modes at portals.* Portals are typically cut into the hillside and preferably expose sound rock. The portal cut is exposed to all of the failure modes of any man-made cut into soil, colluvium, talus, or rock, including slope failure on a discontinuity plane, rock falls, deterioration due to exposure, deep-seated failures, sliding of overburden materials on top of bedrock, etc. Fractures are often opened in the ground due to the excavation, and if filled with rain water, the water pressure can result in failure initiation. Rockfalls can be hazardous to personnel moving in and out of the tunnel. In addition to the typical slope failure phenomena, the portal is also the intersection between the tunnel and the portal cut. Tunnel excavation by blasting, if not carefully controlled, can result in very large overbreaks. For these reasons, the ground surrounding the tunnel must be carefully supported, and the initial tunnel blasting performed with low energy, as discussed in Chapter 5.

b. Failure modes of tunnels and shafts during operation. Most of the modes of failure discussed above apply to the construction environment; once they are dealt with, they pose no further threat. Some of the conditions responsible for the failure modes, however, can also affect long-term performance, especially if they are not dealt with properly. Following are additional modes that apply, typically, to the finished, lined structures.

(1) *Failures due to water pressure.*

(a) Internal water pressure can result in fracture of a concrete lining and escape of the water into the formation. If these formation water pressures cannot dissipate (as in a permeable formation), the formation may be fractured by hydraulic jacking, with the potential for tunnel damage, or worse—instability of adjacent slopes or valley walls. This phenomenon is discussed in Chapter 9. Such failures can occur if the lining is not designed for the hoop tension caused by the internal water pressure and the formation (and formation water) pressure on the exterior is lower than the internal pressure.

(b) The principal failure mode of concern for external water pressure is the buckling of steel-lined tunnels. During operation a steel-lined pressure tunnel is not in danger due to external water pressure, but the empty tunnel must accept the full external pressure without internal balancing pressure. Not infrequently, leakage from the pressure tunnel causes the formation pressure to rise to a value close to that in the tunnel. When the tunnel is then emptied, it has to withstand an external pressure equivalent to the internal pressure.

(c) A tunnel lining is often furnished with an impervious membrane to control groundwater inflow that would otherwise be excessive. As a general rule, this impervious membrane must accept the full external water pressure and be supported by an internal structure capable of withstanding this pressure.

(2) *Tunnel lining failure caused by external loads.*

(a) The failure of a concrete tunnel lining has to be viewed in terms of its functional requirements. A tunnel lining may crack or leak or deteriorate, but as long as it serves its function for the expected lifetime, it has not failed.

(b) The following discussion, for the most part, applies equally to cast-in-place and precast, segmental lining. Tunnel linings in rock are externally contained; they are different from aboveground structures for at least the following reasons:

- Stresses and strains are governed not so much by loads as by interaction between the lining

structure and the ground requiring compatible displacements.

- Except for water pressure, loads on the lining often relax upon displacement and yield; they are not conservative or following loads.

- Radial fractures in a concrete lining do not usually form a mechanism of instability, witness voussoir arches without bonds between blocks. The compressive stress between adjacent blocks combined with friction between the blocks suffices to maintain the stability of the arch, even with a substantial external load.

- Because of net hoop compression (in a circular tunnel, often also for other shapes), a tension fracture from the inside face due to bending does not usually penetrate the thickness of the lining.

- The rock surrounding a tunnel lining is usually under relatively strong compression, and the bond between lining and rock is usually good. Therefore, tendencies to generate external tension fractures due to bending are greatly resisted.

- The usual circular shape is inherently strong and forgiving and, with usual dimensions, resists buckling. Horseshoe and other shapes are not as forgiving.

(c) Structural failure of a concrete lining does occur on occasion. When it does it is usually for one of the following reasons:

- Loss of support around part of the lining due to inadequate concrete placement or contact grouting, especially in the crown of the tunnel, or due to washout of fines, dissolution, or rotting of timber, resulting in uneven loading and support. Unrelieved differential hydrostatic pressures can also exist in such void spaces during filling or emptying of the tunnel.

- Excessive or nonuniform load on a circular lining, causing large distortions, sufficient to create compressive failure in bending (rarely by uniform thrust); nonuniform load may be caused by stratigraphic or structural geologic differences across the tunnel section and by nonuniform swelling or squeezing.

- Excess side pressure on walls of horseshoe-shaped tunnel, resulting in gross bending of the walls or buckling of the floor, or both. This can also result from loss of floor strut due to excessive floor heave.

- External factors, such as effects of adjacent new construction, slope failure at a portal or in the vicinity of the tunnel.

6-4. Seismic Effects on Tunnels, Shafts, and Portals

It is generally acknowledged that underground structures are inherently less sensitive to seismic effects than surface structures. The good performance of underground structures was demonstrated during the 1986 Mexico City earthquake, where subway structures in soft and very soft ground went undamaged and the subway served as the principal lifeline, once power was restored. In contrast, buildings and other surface facilities suffered severe damage. Nonetheless, underground structures can suffer damage in an earthquake under particularly unfavorable conditions. In most cases, however, the vulnerability of a particular structure can be assessed and a design prepared that will eliminate or minimize the effects of earthquakes. The vulnerability of underground structures is examined in Box 6-5.

a. Effect of earthquake shaking on tunnels and shafts.

(1) Earthquake waves traveling through the ground are displacement waves, generally compression (P) or shear (S) waves. Due to scattering and other effects, the seismic displacement waves can vary nearly randomly in space and time. The response of a tunnel or shaft is either axial compression or extension, horizontal or vertical curvature, or ovalizing (racking), or usually a combination of all.

(2) A tunnel or shaft structure subjected to axial and curvature motions may be compared with a beam under combined compression (extension) and bending-maximum and minimum stresses occur at the extremities. The resulting stresses depend on the initial static stresses, upon which the dynamic motion is superimposed.

(3) Ovaling may occur due to a shear wave impinging nearly at a right angle to the tunnel or shaft. While one diameter is increased, the perpendicular diameter is reduced a similar amount, and moments are created

Box 6-5. Case History: Tang-Shan Eearthquake, 1976; Performance of Underground Structures.

The Tang-Shan Earthquake was of Magnitude 7.8, with surface Mercalli intensities of X to XI. It occurred in an industrial area with several coal mines. Surface faulting extended for more than 10 km, and fault traces with displacements up to 1.5 m traversed underground mine facilities. On the surface, destruction was nearly 90-percent complete, and several hundred thousand lives were lost. Damage to underground structures, however, was relatively minor, and all miners, some 1,000 in number, were evacuated safely.

An incline provides access to the Tang-Shan mine, located in the area of greatest surface destruction. The inclined tunnel passes through 4 m of clay and a 62-m strata of limestone before reaching shale and coal strata. The tunnel is horseshoe shaped (arch and straight walls) and lined with bricks or stone blocks, with an unreinforced concrete floor. The tunnel is 1.8-2.5 m high and 1.2-2.5 m wide. Tunnel enlargements for electrical and pumping gear are 2-3 m high and 3-5 m wide. These structures remained essentially intact and passable after the seismic event.

The first 15 m of tunnel through the clay experienced circumferential cracks 1-3 m apart and 10-50 mm wide; a horizontal crack, 20 mm wide, also occurred. Down to a vertical depth of 30 m, the spacing of cracks decreased to more than 10 m, with up to a 10-mm crack width. Beyond this depth, there were occasional cracks. The concrete floor of the pump station at a 30-m depth heaved up to 300 mm and experienced a crack 10 m long, and a few bricks and pieces of plaster loosened and fell. The station at a 230-m depth experienced a floor heave of 200 mm along a length of about 7 m. The station at a 450-m depth showed a 50-mm floor heave in a 1-m area; only small pieces of plaster fell off roof or walls. Damage was noted mostly at weak spots, such as at changes in cross section or lining material, or at bases of arches. There was clearly a great reduction in damage as a function of depth; but on the whole, the tunnel remained intact and passable.

In contrast, pumps and transformers in the underground were damaged; many transformers toppled over. Rail cars tipped on their wheels and lifted up to 30 deg off their rails. People in the mine corridors were thrown into the air up to more than 0.3 m or along horizontally several meters, indicating accelerations greater than one g.

Production drifts in the coal mines, designed and built for a limited lifetime through weak rocks, saw effects such as excessive loading of hydraulic mine struts, breaking of support timber, loosening and fallout of chunks of coal, dust filling the air, squirting of water out of fractures during the earthquake motion, and increased water flow through fractures in general. Most of this behavior occurred within a distance of some 100-150 m from the faults actually observed being displaced. Beyond this range, the mine openings, though violently shaken, showed little permanent damage.

This case history demonstrates the survivability of even poorly supported tunnels and other underground openings through relatively weak rock when subject to violent earthquake motions.

Reference: Wang (1985)

around the opening. Maximum and minimum stresses occur at four points around the opening, at the inside or outside surface of the lining, or tangential to the rock surface in an unlined tunnel.

(4) Regardless of the motion induced by an earthquake, the result is manifested as extension or compression at points around the tunnel or shaft opening. Tensile stresses can occur if the initial tangential stress (usually compression) is small. These transient stresses can usually be considered as pseudo-static superposition on the existing stresses, because the seismic wavelength is almost always much longer than the dimension of the typical underground structure. There is little dynamic amplification, because the resonant frequency of an underground opening is much higher than the typical frequency band of seismic waves. Studies suggest that dynamic stress amplification at the tunnel opening generally gives stresses that can be up to 10 or 15 percent higher than pseudostatic solutions. This is different from typical surface structures (buildings, bridges), whose natural frequency often falls within the typical seismic wave frequency band, and where amplification can be large.

(5) In an unlined tunnel, shaped to have its circumference generally in compression, the additional seismic stresses are generally inconsequential. Blocks of rock that are almost ready to fall can loosen and fall out due to the shaking. Even when tension cracks occur, or existing cracks open, they will typically close again in a fraction of a second, without consequence. Similar arguments apply to a tunnel supported with spot bolts and occasional shotcrete support.

(6) Where a pattern of tensioned bolts has been applied as ground support, the bolts create a compression

ring around the tunnel or cavern arch, preventing tension and holding blocks in place. Similar conditions prevail with untensioned pattern dowels and shotcrete support, where ground motions have induced some tension in the dowels to form a compression arch.

(7) A concrete lining will be subject to compression and extension at points on the exterior and interior of the lining. As discussed in Chapter 9, exterior extension is of no consequence. In the event that tension cracks appear on the interior surface, they will close again after a fraction of a second. Such cracks do not usually extend through the thickness of the concrete and cannot, in themselves, form a failure mechanism. A simplified method of analyzing tunnels in rock for seismic effects is shown in Box 6-6. This simplified method ignores the effect of ground-structure interaction and provides an upper-bound estimate of strains induced in the lining. The method permits a quick verification of the adequacy of the lining design in reasonably competent ground. In very weak ground, ground-structure interaction should be considered to avoid overdesign of the lining.

b. Effects of fault displacement.

(1) Tunnel alignments should avoid active faults whenever possible; however, if faults cannot be avoided, the design must include fault displacement. It is not possible to build a structure that will resist the fault displacement. If the tunnel structure is to remain functional after the earthquake, strategies must be planned to mitigate the effects of fault displacement.

(2) For rail tunnels, the strategy has been to build the tunnel oversized through the fault zone, sufficient to realign the track with acceptable lateral and vertical curves after the event, while reinforcing the ground in and around the shear zone sufficient to prevent collapse. A ground reinforcement system of great ductility is required, such as a combination of lattice girders, wire mesh, rock dowels, and shotcrete. Tunnel damage is expected; however, repairs can be quickly accomplished.

(3) For shallow water tunnels, the most effective solution may be to plan for excavation and replacement of the damaged structure after the event. In a deeper tunnel, repair and replacement may not be so easy. In this case, the tunnel may be oversized through the fault zone and a relatively flexible pipe constructed within the tunnel, providing enough space to avoid shearing the pipe due to the fault motion. The pipe must be supported or suspended to permit motion in any direction.

c. Other permanent displacements of the ground. Portals are particularly vulnerable to permanent displacements during earthquake events. Slope stability in the event of an earthquake can be analyzed using dynamic slope stability analyses, and portal slopes can be reinforced, using tieback anchors or other devices as necessary. Another potential problem is falling rocks, loosened by the earthquake. Large blocks of rock loosening may be secured individually, or shotcrete may be applied to prevent loosening.

Box 6-6. Seismic Analysis of Circular Tunnel Linings (Continued)

1. Longitudinal Bending and Extension or Compression

Obtain seismic input parameter from seismologist:

V_s = maximum particle velocity from shear wave

A_s = maximum particle acceleration from shear wave

Obtain effective shear wave propagation velocity C_s of rock medium from in situ seismic survey or from relationship with effective shear modulus G (under earthquake shear strain level):

$C_s = \sqrt{G/\rho}$

where ρ = specific gravity of rock mass. Shear modulus is related to Young's modulus E_r by

$G = E_r/2(1 + v_r)$

where v_r is Poisson's ratio for the rock mass

With the assumption that the tunnel structure is flexible relative to the ground, then the tunnel structure will conform to the free-field motion of the ground, and the maximum and minimum (compression, extension) strain of the tunnel structure is

$E_{max/min} = \pm(V_s/C_s) \sin\theta \cos\theta \pm (A_sR/C_s^2) \cos^3\theta,$

where R = tunnel radius (strictly speaking, R = distance from extreme compression fiber to neutral axis) and θ = angle of incidence of seismic shear wave. The greatest/smallest strain is usually found for $\theta = 45°$:

$E_{max/min} = \pm 0.5\, V_s/C_s \pm 0.35\, A_sR/C_s^2$ -

2. Ovaling or Racking

A seismic shear wave impinging on a circular tunnel structure at a right angle will cause the structure to rack or ovalize, shortening one diameter D by Δ D and lengthening the orthogonal diameter by an equal amount. In the free field rock mass, the shear strain can be approximated by

$\gamma_{max} = V_s/C_s,$

and an unlined hole driven through the rock mass would suffer an ovalizing distortion of

$\Delta D / D = \pm \gamma_{max} (1 - v_r)$

The maximum strain in the lining, then, is

$E_{max} = V_s/C_s\, [(3(1 - v_r)t/R + 1/2\, R/t\, E_r/E_c \{(1 - v_c^2)/(1 + v_r)\}]$

where t = lining thickness, R = tunnel radius, E_c = concrete modulus, v_c = Poisson's ratio for concrete.

3. Notes

Ovalizing strains are superimposed on strains pre-existing from static loads.
For a maximum earthquake design, usable compressive strain is about 0.003.
Tension cracks due to excessive extension dynamic strains usually cannot be avoided. They will, however, generally close again after the seismic event. Tension cracks can be reduced in size and distributed by appropriate crack reinforcement.

Box 6-6. (Concluded)

4. Example - Los Angeles Metro, Circular Tunnel in San Fernando Formation

$A_s = 0.6g$, $V_s = 3.2$ ft/sec, $C_s = 1360$ ft/sec

$R = 10$ ft, $t = 8.0$ in., $E_c/(1 - v_c^2) = 662{,}400$ ksf, $E_r = 7200$ ksf, $v_r = 0.33$

1. Longitudinal:

$E_{max/min} = \pm 0.5 \times 3.2/1360 \pm 0.35 \times 0.6 \times 32.2 \times 10/1360^2$

$= \pm 0.00118 \pm 0.000037 = \pm 0.00122 < 0.003$ - ok

2. Ovalizing:

$\Delta D/D = + 2 * 3.2/1360 (1 - 0.33) = 0.0031$

$E_{max/min} = \pm 3.2/1360 [3(1 - 0.33)(8/120) \pm 1/2 * 120/8 * 7200/ (1 + 0.33) \times 1/662{,}400)]$

$= \pm 3.2/1360 (0.134 + 0.122) = 0.0006 < 0.003$ - ok

This example is for a concrete tunnel through a weak, soil-like material. Tunnels through stronger, rock-like materials would be subjected to lower seismic strains.

Reference: Wang (1985)

Chapter 7
Design of Initial Support

7-1. Design of Initial Ground Support

a. Initial ground support is installed shortly after excavation in order to make the underground opening safe until permanent support is installed. The initial ground support may also function as the permanent ground support or as a part of the permanent ground support system. The initial ground support must be selected in view of both its temporary and permanent functions.

b. Because of the variability of geologic materials, initial ground support systems are usually not subject to rigorous design but are selected on the basis of a variety of rules. There are three basic methodologies employed in selecting initial ground support, and one or more of these approaches should be used:

- Empirical rules constructed from experience records of satisfactory past performance.

- Theoretical or semitheoretical analysis methods, based on one or more postulated modes of behavior.

- The fundamental approach, involving a definition of potential modes of failure and a selection or design of components to resist these modes of failure.

EM 1110-1-2907 (Rock Reinforcement) and EM 1110-2-2005 (Standard Practice for Shotcrete) provide additional details on these types of ground support.

7-2. Empirical Selection of Ground Support

In past centuries, ground support was always selected empirically. The miner estimated, based on his experience, what timbering was required, and if the timbering failed it was rebuilt stronger. Written rules for selecting ground support were first formulated by Terzaghi (1946). The development of the RQD as a means to describe the character or quality of the rock mass led to correlations between RQD and Terzaghi's rock loads. This development also led to independent ground support recommendations based on RQD. The RQD is also of the basis of two other rock mass characterization schemes used for initial ground support selection, the Geomechanics Classification (Rock Structure Rating (RMR) scheme, Bieniawski 1979), and the Norwegian Geotechnical Institute's Q-system (Barton, Lien, and Lunde 1974). Another classification and ground support selection scheme, the Rock Structure Rating (RSR, Wickham, Tiedemann, and Skinner 1974), is also used.

a. Terzaghi's rock loads and the RQD.

(1) Terzaghi estimated rock loads on steel ribs based on verbal descriptions of the rock mass characteristics. He described the vertical and side loads on the ribs in terms of the height of a loosened mass weighing on the steel rib. The height is a multiple of the width of the tunnel or of the width plus the height. The rock mass descriptions are discussed in Section 3-3. Deere et al. (1970) correlated Terzaghi's rock loads with approximate RQD values and approximate fracture spacings as shown in Table 7-1, and also presented separate ground support recommendations for tunnels excavated conventionally and by TBM as shown in Table 7-2.

(2) Terzaghi's rock load estimates were derived from an experience record that included tunnels excavated by blasting methods and supported by steel ribs or timbers. Ground disturbance and loosening occur due to the blasting prior to installation of initial ground support, and the timber blocking used with ribs permits some displacement of the rock mass. Terzaghi's rock loads generally should not be used in conjunction with methods of excavation and support that tend to minimize rock mass disturbance and loosening, such as excavation on TBM and immediate ground support using shotcrete and dowels. The Deere et al. recommendations are still sound and reasonable, but are now used mainly as a check on other empirical methods.

b. Rock Structure Rating (RSR).

(1) The Rock Structure Rating system was devised by Wickham, Tiedeman, and Skinner in 1972. It was the first published, numerical rating of a rock mass that takes into account a number of geologic parameters and produces a numerical rock load estimate. The geologic parameters considered include the following:

- Rock type.

- Joint pattern (average joint spacing).

- Joint orientations (dip and strike).

- Type of discontinuities.

- Major faults, shears, and folds.

Table 7-1
Terzaghi's Rock Load Classifications as Modified by Deere et al. 1970

Fracture spacing (cm)	RQD (%)	Rock condition	Rock load, H_p Initial	Rock load, H_p Final	Remarks	
		1. Hard and intact	0	0	Generally no side pressure, erratic load changes from point to point	Lining only if spalling or popping
50	98 95	2. Hard stratified or schistose	0	0.25B		Spalling common
	90	3. Massive moderately jointed	0	0.5B		Side pressure if strata inclined, some spalling
20		4. Moderately blocky and seamy	0	0.25B 0.35C		
10	75 50	5. Very blocky, seamy and shattered	0 to 0.6C	0.35B 1.1C		Little or no side pressure
5	25 10 2	6. Completely crushed		1.1C		Considerable side pressure. If seepage, continuous support
2		7. Gravel and sand	0.54C to 1.2C 0.94C to 1.2C	0.62C to 1.38C 1.08C to 1.38C		Dense Side Pressure $P_h = 0.3\gamma(0.5H_t + H_p)$ Loose
Weak and Coherent		8. Squeezing, moderate depth		1.C to 2.1C		Heavy side pressure, continuous support required
Weak and Coherent		9. Squeezing, great depth		2.1C to 4.5C		
		10. Swelling		up to 75 m (250 ft)		Use circular support. In extreme cases: yielding support

Notes:
1. For rock classes 4, 5, 6, 7, when above groundwater level, reduce loads by 50 percent.
2. B is tunnel width; $C = B + H_t$ = width + height of tunnel.
3. γ = density of medium.

Table 7-2
Support Recommendations for Tunnels in Rock (6 m to 12 m diam) Based on RQD (after Deere et al. 1970)

Rock Quality	Tunneling Method	Alternative Support Systems Steel Sets[3]	Rockbolts[3]	Shotcrete
Excellent[1] RQD>90	Boring machine	None to occasional light set. Rock load (0.0-0.2) B	None to occasional	None to occasional local application
	Conventional	None to occasional light set. Rock load (0.0-0.3) B	None to occasional	None to occasional local application 2 to 3 in.
Good[1] 75<RQD<90	Boring machine	Occasional light sets to pattern on 5- to 6-ft center. Rock load (0.0 to 0.4)B	Occasional to pattern on 5- to 6-ft centers	None to occasional local application 2 to 3 in.
	Conventional	Light sets 5- to 6-ft center. Rock load (0.3 to 0.6)B	Pattern, 5- to 6-ft centers	Occasional local application 2 to 3 in.
Fair 50<RQD<75	Boring machine	Light to medium sets, 5- to 6-ft center. Rock load (0.4-1.0)B	Pattern, 4- to 6-ft center	2- to 4-in. crown
	Conventional	Light to medium sets, 4- to 5-ft center. Rock load (0.6-1.3)B	Pattern, 3- to 5-ft center	4-in. or more crown and sides
Poor[2] 25<RQD<50	Boring machine	Medium circular sets on 3- to 4-ft center. Rock load (1.0-1.6)B	Pattern, 3- to 5-ft center	4 to 6 in. on crown and sides. Combine with bolts.
	Conventional	Medium to heavy circular sets on 2- to 4-ft center. Rock load (1.3-2.0)B	Pattern, 2- to 4-ft center	6 in. or more on crown and sides. Combine with bolts.
Very poor[3] RQD<25 (Excluding squeezing or swelling ground)	Boring machine	Medium to heavy circular sets on 2-ft center. Rock load (1.6 to 2.2)B	Pattern, 2- to 3-ft center	6 in. or more on whole section. Combine with medium sets.
	Conventional	Heavy circular sets on 2-ft center. Rock load (1.6 to 2.2)B	Pattern, 3-ft center	6 in. or more on whole section. Combine with medium sets.
Very poor[3] (Squeezing or swelling)	Boring machine	Very heavy circular sets on 2-ft center. Rock load up to 250 ft.	Pattern, 2- to 3-ft center	6 in. or more on whole section. Combine with heavy sets.
	Conventional	Very heavy circular sets on 2-ft center. Rock load up to 250 ft.	Pattern, 2- to 3-ft center	6 in. or more on whole section Combine with heavy sets.

Notes:
[1] In good and excellent rock, the support requirement will be, in general, minimal but will be dependent upon joint geometry, tunnel diameter, and relative orientations of joints and tunnel.
[2] Lagging requirements will usually be zero in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.
[3] Mesh requirements usually will be zero in excellent rock and will range from occasional mesh (or strips) in good rock to 100-percent mesh in very poor rock.
[4] B = tunnel width.

- Rock material properties.
- Weathering and alteration.

(2) Some of these are combined in various ways. The construction parameters are size of tunnel, direction of drive (relative to discontinuities), and method of excavation. All of these parameters are combined as shown in Table 7-3; the RSR value is the sum of parameters A, B, and C. With the assumption that TBM excavation causes less disturbance, the RSR value is adjusted by the factor shown on Figure 7-1 as a function of tunnel size.

(3) Predicted tunnel arch rock loads in kips per square foot as a function of RSR and tunnel width or diameter are shown on Figure 7-2.

Table 7-3
Rock Structure Rating - Parameter A: General Area Geology (after Wickham et al. 1974)

	Basic Rock Type				Geological Structure			
	Hard	Med.	Soft	Decomp.	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Rock Structure Rating - Parameter B: Joint Pattern, Direction of Drive (after Wickham et al. 1974)

Average joint spacing	Strike ⊥ to axis					Strike ∥ to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against Dip		Both		
	Dip of prominent joints[1]					Dip of prominent joints[1]		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed < 2 in.	9	11	13	10	12	9	9	7
2. Closely jointed 2-6 in.	13	16	19	15	17	14	14	11
3. Moderately jointed 6-12 in.	23	24	28	19	22	23	23	19
4. Moderate to blocky 1-2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive 2-4 ft	36	38	40	33	35	36	34	28
6. Massive > 4 ft	40	43	45	37	40	40	38	34

Rock Structure Rating - Parameter C: Groundwater, Joint Condition (after Wickham et al. 1974)

Anticipated water inflow (gpm/1,000 ft)	Sum of parameters *A* + *B*					
	13-44			45-75		
	Joint condition[2]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight < 200 gpm	19	15	9	23	19	14
Moderate 200-1,000 gpm	15	11	7	21	16	12
Heavy > 1,000 gpm	10	8	6	18	14	10

[1] Dip: flat: 0-20 deg; dipping: 20-50 deg; and vertical: 50-90 deg.
[2] Joint condition: Good = tight or cemented; Fair = slightly weathered or altered; Poor = severely weathered, altered, or open.

(4) The RSR database consists of 190 tunnel cross sections, of which only three were shotcrete supported and 14 rock bolt supported; therefore, the database only supports rock load recommendations for steel ribs.

c. *Geomechanics Classification (RMR System).*

(1) This system, developed by Bieniawski (1979), uses the following six parameters:

- Uniaxial compressive strength of rock.
- RQD.
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater condition.
- Orientation of discontinuities.

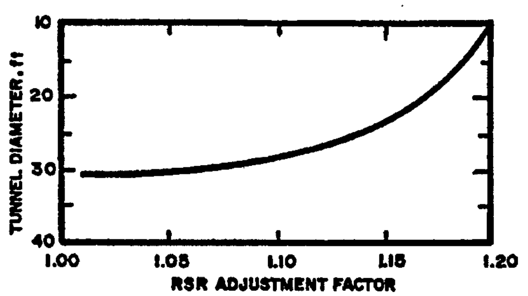


Figure 7-1. RSR adjustment factor for TBM excavation

The components of this classification system are shown in Table 7-4. Part A of this table shows the five basic parameters and their ranges as dependent on the rock mass condition. Together, the rating numbers for the five parameters add up to the basic RMR value. Part B gives a rating adjustment based on the orientation of the discontinuities relative to the tunnel orientation. The effect of strike and dip on tunneling is shown in Table 7-5. Part C of Table 7-4 shows the general classification of the rock mass based on RMR, ranging from very good to very poor rock. Part D presents some numerical predictions of stand-up time, rock mass cohesion, and friction based on RMR. Unal (1983) presented the following equation for the ground load, measured as the rock load height:

$$H_b = (1 - RMR/100)\ B$$

where B is the tunnel width. Recommendations for excavation and support for a 10-m-wide tunnel excavated by blasting are presented in Table 7-6.

(2) Other correlations using RMR have been developed. Figure 7-3 shows a correlation between RMR and the in situ modulus of deformation of the rock mass. Serafin and Pereira (1983) produced a different correlation, applicable also for RMR < 50:

$$E_M = 10\ (RMR/40 - 0.25)$$

(3) The RMR system is based on a set of case histories of relatively large tunnels excavated using blasting. Ground support components include rock bolts (dowels), shotcrete, wire mesh, and for the two poorest rock classes, steel ribs. The system is well suited for such conditions but not for TBM-driven tunnels, where rock damage is less and where immediate shotcrete application may not be feasible.

d. The Q-System for rock mass classification.

(1) The NGI Q-System (Barton, Lien and Lunde 1974) is generally considered the most elaborate and the most detailed rock mass classification system for ground support in underground works. The value of the rock quality index Q is determined by

$$Q = (RQD/J_n)\ (J_r/J_a)\ (J_w/SRF)$$

where

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

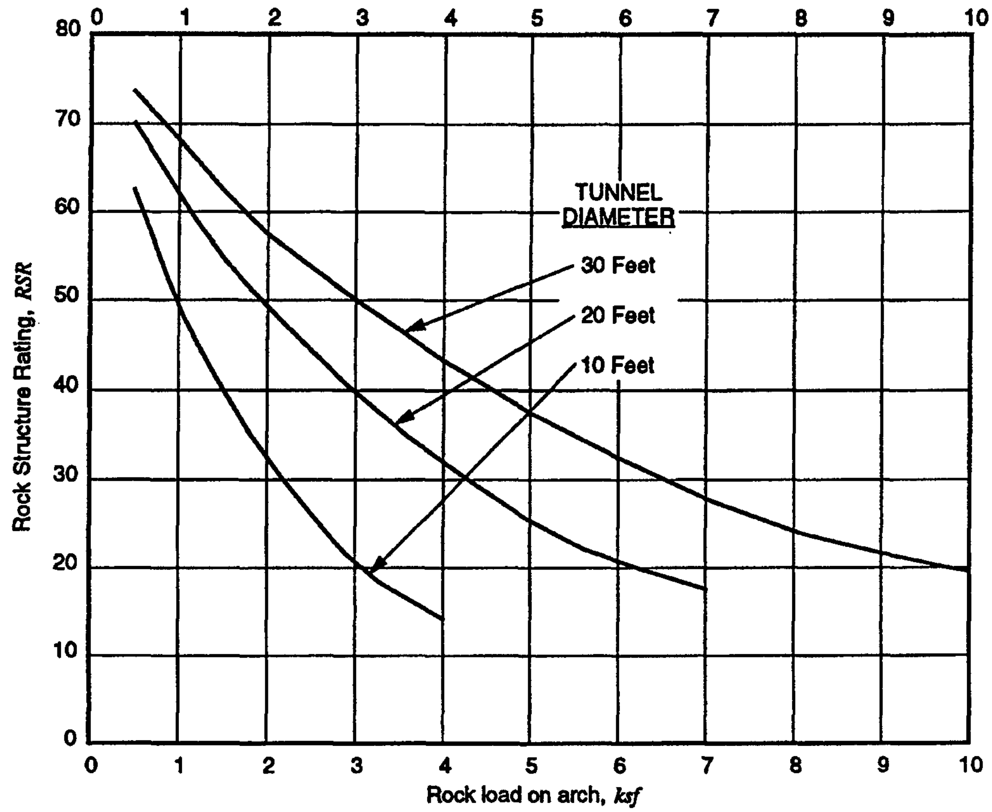
J_w = joint water reduction factor

SRF = stress reduction factor

The numerical values of these numbers are determined as described in Table 7-7.

(2) To relate the Q-value to ground support requirements, an equivalent dimension is defined as the width of the underground opening, divided by the excavation support ratio (ESR). The value of the ESR depends on the ultimate use of the underground opening and the time of exposure; the following values of ESR are recommended:

- ESR = 3-5 for temporary mine openings.
- ESR = 2-2.5 for vertical shafts (highest for circular).
- ESR = 1.6 for permanent mine openings, hydropower water tunnels (except high-pressure tunnels), and temporary works, including tunnels where a final lining is later placed.
- ESR = 1.3 for minor traffic tunnels, surge chambers, access tunnels.
- ESR = 1.0 for most civil works, including power stations, major traffic tunnels, water pressure tunnels, intersections of tunnels, and portals.



Correlation of Rock Structure Rating to Rock Load and Tunnel Diameter

Tunnel Diameter (D)	(Wr) Rock Load on Tunnel Arch (k/sq ft)											
	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
	Corresponding Values of Rock Structure Ratings (RSR)											
10'	62.5	49.9	40.2	32.7	21.6	13.8						
12'	65.0	53.7	44.7	37.5	26.6	18.7						
14'	66.9	56.6	48.3	41.4	30.8	22.9	16.8					
16'	68.3	59.0	51.2	44.7	34.4	26.6	20.4	15.5				
18'	69.5	61.0	53.7	47.6	37.6	29.9	23.8	18.8				
20'	70.4	62.5	55.7	49.9	40.2	32.7	26.6	21.6	17.4			
22'	71.3	63.9	57.5	51.9	42.7	35.3	29.3	24.3	20.1	16.4		
24'	72.0	65.0	59.0	53.7	44.7	37.5	31.5	26.6	22.3	18.7		
26'	72.6	66.1	60.3	55.3	46.7	39.6	33.8	28.8	24.6	20.9	17.7	
28'	73.0	66.9	61.5	56.6	48.3	41.4	35.7	30.8	26.6	22.9	19.7	16.8
30'	73.4	67.7	62.4	57.8	49.8	43.1	37.4	32.6	28.4	24.7	21.5	18.6

Figure 7-2. Tunnel arch load as a function of RSR and tunnel diameter

Table 7-4
Geomechanics Classification of Jointed Rock Masses

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
			>250 MPa	100-150 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90-100%	75-90%	50-75%	25-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6-2 m	200-600 mm	60-200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continu-ous. No separation. Unweathered wall rock.	Slightly rough surfaces. Separation <1 mm. Slightly weathered walls.	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls.	Slickensided surfaces OR Gouge < 5 mm thick. Separation 1-5 mm. Continuous.	Soft gouge > 5 mm thick OR Separation > 5 mm. Continuous.		
	Rating		30	25	20	10	0		
5	Ground-water	Inflow per 10 m tunnel length	None	<10 L/min	10-25 L/min	25-125 L/min	>125 L/min		
		Ratio: joint water pressure major principal stress	OR 0	OR 0.0-0.1	OR 0.1-0.2	OR 0.2-0.5	OR >0.5		
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations and dips		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	41 ← 21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15-m span	6 months for 8-m span	1 week for 5-m span	10 hr for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass	>400 kPa	300-400 kPa	200-300 kPa	100-200 kPa	<100 kPa
Friction angle of the rock mass	>45°	35-45°	25-45°	15-25°	<15°

Table 7-5
Effect of Discontinuity Strike and Dip Orientations in Tunneling

Strike perpendicular to tunnel axis Drive with dip			
Dip 45-90°	Dip 20-45°	Dip 45-90°	Dip 20-45°
Strike parallel to tunnel axis			Irrespective of strike
Dip 20-45°	Dip 45-90°		Dip 0-20°
Fair	Very Unfavorable		Fair

Table 7-6
Geomechanics Classification Guide for Excavation and Support in Rock Tunnels After Bieniawski (1979)

SHAPE: HORSESHOE; WIDTH: 10 M; VERTICAL STRESS: BELOW 25 MPa; CONSTRUCTION: DRILLING AND BLASTING

Rock Mass Class	Excavation	Rock Bolts (20 mm diam., fully bonded)	Shotcrete	Steel Sets
Very good rock, I RMR:81-100	Full face 3-m advance.	Generally no support required except for occasional spot bolting.		
Good rock, II RMR:61-80	Full face 1.0- to 1.5-m advance. Complete support 20 m from face.	Locally bolts in crown 3 mm long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None
Fair rock, III RMR:41-60	Top heading and bench 1.5- to 3-m advance in top heading. Commerce support after each	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	None
Poor rock, IV RMR:21-40	Top heading and bench 1.0- to 1.5-m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
Very poor rock, V RMR: <20	Multiple drifts. 0.5- to 1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

- ESR = 0.8 for underground railroad stations, sports arenas, and similar public areas.

(3) For application to initial support, where a final lining is placed later, multiply the ESR value by 1.5. The following correlations apply, albeit with considerable variation:

- Maximum unsupported span = 2 ESR $Q^{0.4}$ (m).

- Permanent support pressure, with three or more joint sets: $P = 2.0\ Q^{-1/3}/Jr$.

- Permanent support pressure, with less than three joint sets: $P = 2.0\ J_n^{1/2}Q^{-1/3}/3J_r$.

(4) Barton, Lien, and Lunde (1974) provide 38 support categories (see Figure 7-4) with detailed support recommendations, as enumerated in the annotated Table 7-8.

(5) With all of the commentaries accompanying the tables, the Q-system works very much like an expert system. A careful examination of all the commentaries reveals that the system incorporates features of rock behavior not entirely evident from the basic parameters. This adds to the flexibility and range of application of the system.

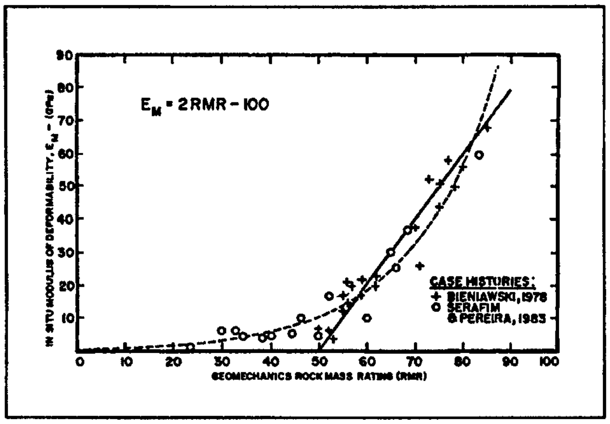


Figure 7-3. Correlation between in situ modulus of deformation and RMR

(6) The Q-system is derived from a database of underground openings excavated by blasting and supported by rock bolts (tensioned and untensioned), shotcrete, wire and chain-link mesh, and cast-in-place concrete arches. For TBM-driven tunnels, it is recommended that the Q-value should be increased by a factor of 5.0.

e. Restrictions in the use of empirical ground support selection systems.

(1) The empirical methods of ground support selection provide a means to select a ground support scheme based on facts that can be determined from explorations, observations, and testing. They are far from perfect and can sometimes lead to the selection of inadequate ground support. It is therefore necessary to examine the available rock mass information to determine if there are any applicable failure modes not addressed by the empirical systems.

(2) A major flaw of all the empirical systems is that they lead the user directly from the geologic characterization of the rock mass to a recommended ground support without the consideration of possible failure modes. A number of potential modes of failure are not covered by some or all of the empirical methods and must be considered independently, including the following:

- Failure due to weathering or deterioration of the rock mass.
- Failure caused by moving water (erosion, dissolution, excessive leakage, etc.).
- Failure due to corrosion of ground support components.
- Failure due to squeezing and swelling conditions.
- Failure due to overstress in massive rock.

(3) The empirical systems are largely based on blasted tunnels and produce ground support recommendations that are a function of the age of the empirical system. System recommendations should be reinterpreted based on current methods of excavation. For example, TBM tunneling produces a favorable tunnel shape and a minimum of ground disturbance; however, the application of shotcrete close to the tunnel face is difficult. Therefore, substitutes for shotcrete, including dowels with wire mesh, ribs with wire mesh, or precast segments, must be applied.

(4) Similarly, new ground support methods and components must be considered. For example, the use of steel fiber reinforced shotcrete, friction dowels, lattice girders, or segmental concrete linings are not incorporated in the empirical systems.

7-3. Theoretical and Semitheoretical Methods

Most theoretical methods of design for rock bolts, dowels, or shotcrete are based on certain assumptions regarding the configuration of discontinuities.

a. Rock bolt analyses.

(1) The simplest methods of rock bolt analysis are the wedge analyses, where the stability of a wedge is analyzed using two- or three-dimensional equilibrium equations. Examples are shown in Figure 7-5. These types of analysis are useful when directions of discontinuities are known and can show which wedges are potentially unstable and indicate the appropriate orientation of bolts or dowels for their support.

(2) For a flat roof in a horizontally layered rock (Figure 7-6), Lang and Bischoff (1982) developed an analysis to show the effect of rock bolts. If the rock bolts are tensioned, either by active tensioning or passively by ground movements, a horizontal compressive stress develops within the zone of the bolts. This enables the beam consisting of the layers of rock tied together to carry a moment, and the edge of the beam to carry a shear load. Thus, the reinforced rock stays suspended. In a similar manner, bolts installed around an arch will increase the

Table 7-7
Input Value to Estimate of Q

1. ROCK QUALITY DESIGNATION (RQD)

A. Very poor	0 - 25
B. Poor 25 - 50	
C. Fair	50 - 75
D. Good	75 - 90
E. Excellent	90 - 100

Note: (i) Where RQD is reported or measured as <10 (including 0), a nominal value of 10 is used to evaluate Q in equation (1)
(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate

2.	JOINT SET NUMBER	(J_n)
A.	Massive, none or few joints	0.5 - 1.0
B.	One joint set	2
C.	One joint set plus random	3
D.	Two joint sets	4
E.	Two joint sets plus random	6
F.	Three joint sets	9
G.	Three joint sets plus random	12
H.	Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15
J.	Crushed rock, earthlike	20

Note: (i) For intersections use (3.0 x J_n)

Note: (ii) For portals use (2.0 J_n)

3. JOINT ROUGHNESS NUMBER

(a) *Rock wall contact* and
(b) *Rock wall contact before* 100-*mm shear* (J_r)

A.	Discontinuous joints	4
B.	Rough or irregular, undulating	3
C.	Smooth, undulating random	2
D.	Slickensided, undulating	1.5
E.	Rough or irregular, planar	1.5
F.	Smooth, planar	1.0
G.	Slickensided, planar	0.5

Note: (i) Descriptions refer to small-scale features and intermediate scale features, in that order.

(c) *No rock wall contact when sheared*

H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J.	Sandy, gravelly, or crushed, some thick enough to prevent rock wall contact	1.0

Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
(iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength

(Sheet 1 of 3)

Table 7-7. (Continued)

4.	JOINT ALTERATION NUMBER	(J_a)	ϕ_r
A.	Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote	0.75	(-)
B.	Unaltered joint walls, surface staining only.	1.0	(25-35°)
C.	Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	(25-30°)
D.	Silty- or sandy-clay coatings, small clay fraction (nonsoft.)	3.0	(20-25°)
E.	Softening or low-friction clay mineral coatings, i.e., kaolinite or mica. Also, chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays..	4.0	(8-16°)
	(b) *Rock wall contact before 100-mm shear*		
F.	Sandy particles, clay-free disintegrated rock, etc..	4.0	(25-30°)
G.	Strongly overconsolidated nonsoftening clay mineral fillings (continuous, but <5-mm thickness).	6.0	(16-24°)
H.	Medium or low overconsolidation, softening, clay-mineral fillings (continuous but <5-mm thickness).	8.0	(12-16°)
J.	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5-mm thickness) Value of Ja depends on percent of swelling clay-size particles and access to water, etc.	8 - 12	(6-12°)
	(c) *No rock wall contact when sheared*		
K.L. M.	Zones or bands of disintegrated or crushed rock and clay (see G,H,J for description of clay condition)	6, 8 or 8-12	(6-24°)
N.	Zones or bands of silty- or sandy-clay, small clay fraction (nonsoftening).	5.0	(-)
O.P. R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition	10, 13, or 13-20	(6-24°)
5.	JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pres. (kPa^2)
A.	Dry excavations or minor inflow, i.e., <5 ℓ/min. locally	1.0	<100
B.	Medium inflow or pressure, occasional outwash of joint fillings	0.66	100-250
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	250-1,000
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	250-1,000
E.	Exceptionally high inflow or water pressure at blasting, decaying with time.	0.2-0.1	>1,000
F.	Exceptionally high inflow or water pressure continuing without noticeable decay.	0.1-0.05	>1,000

Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
(ii) Special problems caused by ice formation are not considered.

(Sheet 2 of 3)

Table 7-7 (Concluded)

6. STRESS REDUCTION FACTOR

(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.*

		(SRF)
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤50 m)	5
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)	2.5
D.	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E.	Single shear zones in competent rock (clay-free) (depth of excavation ≤50 m)	5.0
F.	Single shear zones in competent rock (clay-free) (depth of excavation >50 m)	2.5
G.	Loose open joints, heavily jointed or "sugar cubes," etc. (any depth)	5.0

Note: (i) Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.

(b) *Competent rock, rock stress problems*

		σ_c/σ_1	σ_c/σ_1	(SRF)
H.	Low stress, near surface	>200	>13	2.5
J.	Medium stress	200-10	13-0.66	1.0
K.	High stress, very tight structure (usually favorable to stability, may be unfavorable for wall stability)	10-5	0.66-0.33	0.5-2
L.	Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M.	Heavy rock burst (massive rock)	<2.5	<0.16	10-20

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where : σ_c = unconfined compression strength, and σ_t = tensile strength (point load), and σ_1 and σ_3 are the major and minor principal stresses.

(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

(c) *Squeezing rock:plastic flow of incompetent rock under the influence of high rock pressure*

		(SRF)
N.	Mild squeezing rock pressure	5-10
O.	Heavy squeezing rock pressure	10-20

(d) *Swelling rock:chemical swelling inactivity depending on presence of water*

P.	Mild squeezing rock pressure	5-10
R.	Heavy squeezing rock pressure	10-15

(Sheet 3 of 3)

level of confinement in the zone of the bolts (see Figure 7-7), thus increasing the effective compressive strength of the material in the arch.

(3) Analyses of this type led Lang (1961) to formulate his empirical rules for rock bolt design, reproduced as Table 7-9. This table applies to ground conditions that

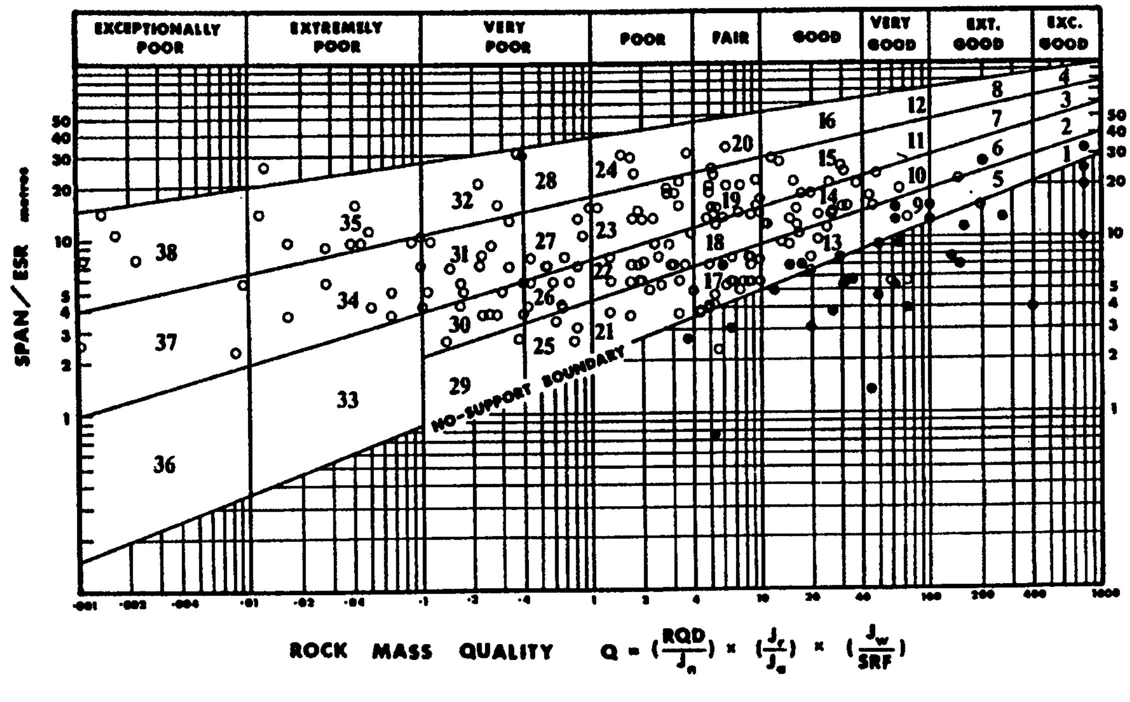


Figure 7-4. Rock support categories shown by box numbers, see Table 7-8

require more than spot bolting for ground support. Where joint spacings are so close that raveling between rock bolts is likely, the rock bolt pattern must be supplemented with wire mesh, shotcrete, or fiber-reinforced shotcrete.

b. Shotcrete analyses.

(1) The function of shotcrete in tunnel construction is to create a semistiff immediate lining on the excavated rock surface. The shotcrete must have a high initial strength for good bond to the rock surface and a high degree of ductility and toughness to absorb and block ground movement. The shotcrete, by its capacity to accept shear and bending and its bond to the rock surface, prevents the displacement of blocks of rock that can potentially fall. Shotcrete also can act as a shell and accept radial loads. It is possible to analyze all of these modes of failure only if the loads and boundary conditions are known.

(2) With the "falling block theory," the weight of a wedge of rock is assumed to load the skin of shotcrete, which can then fail by shear, diagonal tension, bonding loss, or bending (see Figure 7-8). Given the dimensions of the falling block and properties of the shotcrete, it is possible to determine the required thickness of shotcrete, using standard structural calculations.

(3) With the "arch theory," an external load is assumed, and the shotcrete shell is analyzed as an arch, with bending and compression. Where the shotcrete is held by anchors and loaded between the anchors, it may be analyzed either as a circular slab held by the anchor in the middle or as a one-way slab between rows of anchors.

(4) Neither the falling-block or the arch theory can be expected to provide anything more than crude approximations of stresses in the shotcrete, considering the dynamic environment of fresh shotcrete. When shotcrete is used in the method of sequential excavation and support such as NATM, it is possible to reproduce the construction sequence by computer analyses, including the effect of variations of shotcrete modulus and strength with time. In this fashion it is possible to estimate the load buildup in the shotcrete lining as the ground yields to additional excavation and as more layers of shotcrete are applied.

Table 7-8
Ground Support Recommendation Based on Q

Support Category	*Conditional Factors* $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of Support	Notes
1*	-	-	-	sb(utg)	-
2*	-	-	-	sb(utg)	-
3*	-	-	-	sb(utg)	-
4*	-	-	-	sb(utg)	-
5*	-	-	-	sb(utg)	-
6*	-	-	-	sb(utg)	-
7*	-	-	-	sb(utg)	-
8*	-	-	-	sb(utg)	-
Note:	The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth-wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single application of shotcrete, especially where the excavation height is ≥25 m. Future case records should differentiate categories 1 to 8.				
9	≥20	-	-	sb(utg)	-
	<20	-	-	B(utg) 2.5-3 m	-
10	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(utg) 1.5-2 m	-
				+clm	
11*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m	-
				+clm	
12*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m	-
13	≥10	≥1.5	-	sb(utg)	I
	≥10	<1.5	-	B(utg) 1.5-2 m	I
	<10	≥1.5	-	B(utg) 1.5-2 m	I
	<10	<1.5	-	B(utg) 1.5-2 m	I
				+S 2-3 cm	
14	≥10	-	≥15	B(tg) 1.5-2 m	I. II
				+clm	
	<10	-	≥15	B(tg) 1.5-2 m	I. II
				+S(mr) 5-10 cm	
	-	-	<15	B(utg) 1.5-2 m	I. III
15	>10	-	-	B(tg) 1.5-2 m	I. II. IV
				+clm	
	≤10	-	-	B(tg) 1.5-2 m	I. II. IV
				+S(mr) 5-10 cm	
16*	>15	-	-	B(tg) 1.5-2 m	I. V. VI
See				+clm	
note	≤15	-	-	B(tg) 1.5-2 m	I. V. VI
XII		-	-	+S(mr) 10-15 cm	
17	≥30	-	-	sb(utg)	I
	≥10, ≤30	-	-	B(utg) 1-1.5 m	I
	≤10	-	≥6 m	B(utg) 1-1.5 m	I
				+S 2-3 cm	
	<10	-	<6 m	S 2-3 cm	I

(Sheet 1 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of Support	Notes
18	>5	-	≥10 m	B(tg) 1-1.5 m +clm	I. III
	>5	-	<10 m	B(utg) 1-1.5 m +clm	I
	≤5	-	≥10 m	B(tg) 1-1.5 m +S 2-3 cm	I. III
	≤5	-	<10 m	B(utg) 1-1.5 m +S 2-3 cm	I
19	-	-	>20 m	B(tg) 1-2 m +S(mr) 10-15 cm	I. II. IV.
	-	-	<20 m	B(tg) 1-1.5 m +S(mr) 5-10 cm	I.II
20* See note XII	-	-	>35	B(tg) 1-2 m +S(mr) 20-25 cm	I. V. VI
	-	-	<35 m	B(tg) 1-2 m +S(mr) 10-20 cm	I.II.IV
21	≥12.5	≤0.75	-	B(utg) 1 m +S 2-3 cm	I
	<12.5	≤0.75	-	S 2.5-5 cm	I
	-	>0.75	-	B(utg) 1 m	I
22	>10, <30	>1.0	-	B(utg) 1 m +clm	I
	≤10	>1.0	-	S 2.5-7.5 cm	I
	<30	≤1.0	-	B(utg) 1 m +S(mr) 2.5-5 cm	I
	≥30	-	-	B(utg) 1 m	I
23	-	-	≥15 m	B(tg) 1-1.5 m +S(mr) 10-15 cm)	I. II. IV VII
	-	-	<15 m	B(utg) 1-1.5 m +S(mr) 5-10 cm	I
24* See note XII	-	-	≥30 m	B(tg) 1-1.5 m +S(mr) 15-30 cm	I. V. VI.
	-	-	<30 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I. II. IV
25	>10	>0.5	-	B(utg) 1 m + mr or clm	I
	≤10	>0.5	-	B(utg) 1 m +S(mr) 5 cm	I
	-	≤0.5	-	B(tg) 1 m +S(mr) 5 cm	I
26	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII. X. XI
	-	-	-	B(tg) 1 m +S 2.5-5 cm	I. IX

(Sheet 2 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of Support	Notes
27	-	-	≥12 m	B(tg) 1 m +S(mr) 7.5-10 cm	I. IX
	-	-	<12 m	B(utg) 1 m +S(mr) 5-7.5 cm	I. IX
	-	-	>12 m	CCA 20-40 cm +B(tg) 1 m	VIII. X. XI
	-	-	<12 m	S(mr) 10-20 cm +B(tg) 1 m	VIII. X. XI
28* See note XII	-	-	≥30 m	B(tg) 1 m +S(mr) 30-40 cm	I. IV. V. IX
	-	-	≥20, <30 m	B(tg) 1 m +S(mr) 20-30 cm	I. II. IV. IX
	-	-	<20 m	B(tg) 1 m +S(mr) 15-20 cm	I. II. IX
	-	-	-	CCA(sr) 30-100 cm +B(tg) 1 m	IV. VIII. X. XI
29*	>5	0.25	-	B(utg) 1 m +S 2-3 cm	-
	≤5	>0.25	-	B(utg) 1 m +S(mr) 5 cm	-
	-	≤0.25	-	B(tg) 1 m +S(mr) 5 cm	-
30	≥5	-	-	B(tg) 1 m +S 2.5-5 cm	IX
	<5	-	-	S(mr) 5-7.5 cm	IX
	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII. X. XI
31	>4	-	-	B(tg) 1 m +S(mr) 5-12.5 cm	IX
	≤4,≥1.5	-	-	S(mr) 7.5-25 cm	IX
	≤1.5	-	-	CCA 20-40 cm +B(tg) 1 m	IX. XI.
	-	-	-	CCA(Sr) 30-50 cm +B(tg) 1 m	VIII. X. XI.
32 See note XII	-	-	≥20 m	B(tg) 1 m +S(mr) 40-60 cm	II. IV. IX. XI
	-	-	<20m	B(tg) 1 m +S(mr) 20-40 cm	III. IV. XI. IX.
	-	-	-	CCA(sr) 40-120 cm +B(tg) 1 m	IV. VIII. X. XI
33*	≥2	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	IX
	<2	-	-	S(mr) 5-10 cm	IX
	-	-	-	S(mr) 7.5-15 cm	VIII. X
34	≥2	≥0.25	-	B(tg) 1 m +S(mr) 5-7.5 cm	IX
	<2	≥0.25	-	S(mr) 7.5-15 cm	IX
	-	≤0.25	-	S(mr) 15-25 cm	IX
	-	-	-	CCA(sr) 20-60 cm +B(tg) 1 m	VIII. X. XI

(Sheet 3 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* RQD / J_n	J_r / J_a	SPAN / ESR	Type of Support	Notes
	-	-	≥15 m	B(tg) 1 m +S(mr) 30-100 cm	II. IX. XI
35 See note XII	-	-	≥15 m	CCA(sr) 60-200 cm +B(tg) 1 m	VIII.X. XI. II
	-	-	<15 m	B(tg) 1 m +S(mr) 20-75 cm	IX. III. XI.
			<15 m	CCA(sr) 40-150 cm +B(tg) 1 m	VII. X. XI. III
	-	-	-	S(mr) 10-20 cm	IX
36*	-	-	-	S(mr) 10-20 cm +B(tg) 0.5-1.0 m	VIII. X. XI
37	-	-	-	S(mr) 10-20 cm	IX
				S(mr) 20-60 cm +B(tg) 0.5-1.0 m	VIII. X. XI
			≥10 m	CCA(sr)100-300 cm	IX
38 See note XIII	-	-	≥10 m	CCA(sr)100-300 cm +B(tg) 0.5-1.0 m	VIII. X. II. XI
	-	-	<10 m	S(mr) 70-200 cm	IX
	-		<10 m	S(mr) 70-200 cm +B(tg) 1 m	VII. X. III. XI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

Key to Support Tables:

sb	=	spot bolting
B	=	systematic bolting
(utg)	=	untensioned, grouted
(tg)	=	tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S	=	shotcrete
(mr)	=	mesh reinforced
clm	=	chain link mesh
CCA	=	cast concrete arch
(sr)	=	steel reinforced

Supplementary Notes by BARTON, LIEN *and* LUNDE

I. For cases of heavy bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i.e., 3, 5, and 7 m.

III. Several bolt lengths often used in same excavation, i.e., 2, 3, and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2-4 m.

V. Several bolt lengths often used in same excavation, i.e., 6, 8, and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4-6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain-link mesh, and a free span concrete arch roof (25-40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

(Sheet 4 of 5)

Table 7-8 (Concluded)

IX.	Cases not involving swelling clay or squeezing rock.
X.	Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.
XI.	According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., $RQD/J_n < 1.5$, for example, a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete, but it may not be effective when $RQD/J_n < 1.5$ or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick-setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.
XII.	For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR > 15 m only).
XIII.	Multiple drift method usually needed during excavation and support of arch, walls, and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).
Supplementary notes by HOEK *and* BROWN *(1980)*	
A.	Chain-link mesh is sometimes used to catch small pieces of rock that can become loose with time. It should be attached to the rock at intervals of between 1 and 1.5 m, and short grouted pins can be used between bolts. Galvanized chain-link mesh should be used where it is intended to be permanent, e.g., in an underground powerhouse.
B.	Weldmesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chain-link mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires and air pockets are formed with consequent rusting of the wire. When choosing weldmesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2-mm wires set at 100-mm intervals (designated 100 by 100 by 4.2 weldmesh) are used for reinforcing shotcrete.
C.	In poorer quality rock, the use of untensioned grouted dowels as recommended by Barton, Lien, and Lunde (1974) depends upon immediate installation of these reinforcing elements behind the face. This depends upon integrating the support drilling and installation into the drill-blast-muck cycle, and many non-Scandinavian contractors are not prepared to consider this system. When it is impossible to ensure that untensioned grouted dowels are going to be installed immediately behind the face, consideration should be given to using tensioned rock bolts that can be grouted at a later stage. This ensures that support is available during the critical excavation stage.
D.	Many contractors would consider that a 200-mm-thick cast concrete arch is too difficult to construct because there is not enough room between the shutter and the surrounding rock to permit easy access for placing concrete and using vibrators. The USACE has historically used 10 in. (254 mm) as a normal minimum, while some contractors prefer 300 mm.
E.	Barton, Lien, and Lunde (1974) suggest shotcrete thicknesses of up to 2 m. This would require many separate applications, and many contractors would regard shotcrete thicknesses of this magnitude as both impractical and uneconomical, preferring to cast concrete arches instead. A strong argument in favor of shotcrete is that it can be placed very close to the face and hence can be used to provide early support in poor quality rock masses. Many contractors would argue that a 50- to 100-mm layer is generally sufficient for this purpose, particularly when used in conjunction with tensioned rock bolts as indicated by Barton, Lion, and Lunde (1974) and that the placing of a cast concrete lining at a later stage would be a more effective way to tackle the problem. Obviously, the final choice will depend upon the unit rates for concreting and shotcreting offered by the contractor and, if shotcrete is cheaper, upon a practical demonstration by the contractor that he can actually place shotcrete to this thickness.
	In North America, the use of concrete or shotcrete linings of up to 2 m thick would be considered unusual, and a combination of heavy steel stets and concrete would normally be used to achieve the high support pressures required in very poor ground.
Supplementary note	
	Untensioned, grouted rock bolts are recommended in several support categories. At the time when Barton, Lien, and Lunde proposed their guide for support measures, the friction-anchored rock bolts were not yet available. Under appropriate circumstances, friction dowels are relatively inexpensive alternatives for initial, temporary ground-support application.

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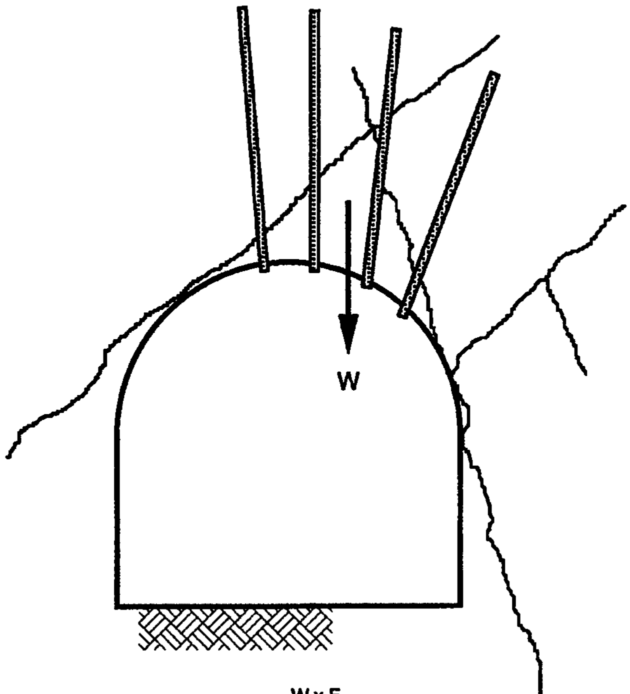
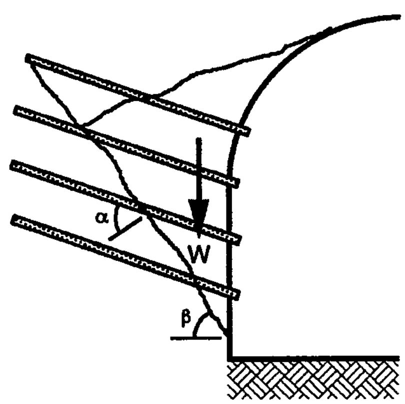


$$N = \frac{W \times F}{B}$$

N = Number of bolts (dowels)
W = Weight of wedge
F = Safety factor (1.5 to 3.0)
φ = Friction angle of sliding surface
c = Cohesion of sliding surface
A = Area of sliding surface
B = Load bearing capacity of bolt (dowel)



$$N = \frac{W\,(F \sin\beta - \cos\beta \tan\varphi) - cA}{B\,(\cos\alpha \tan\varphi + F \sin\alpha)}$$

Figure 7-5. Gravity wedge analyses to determine anchor loads and orientations

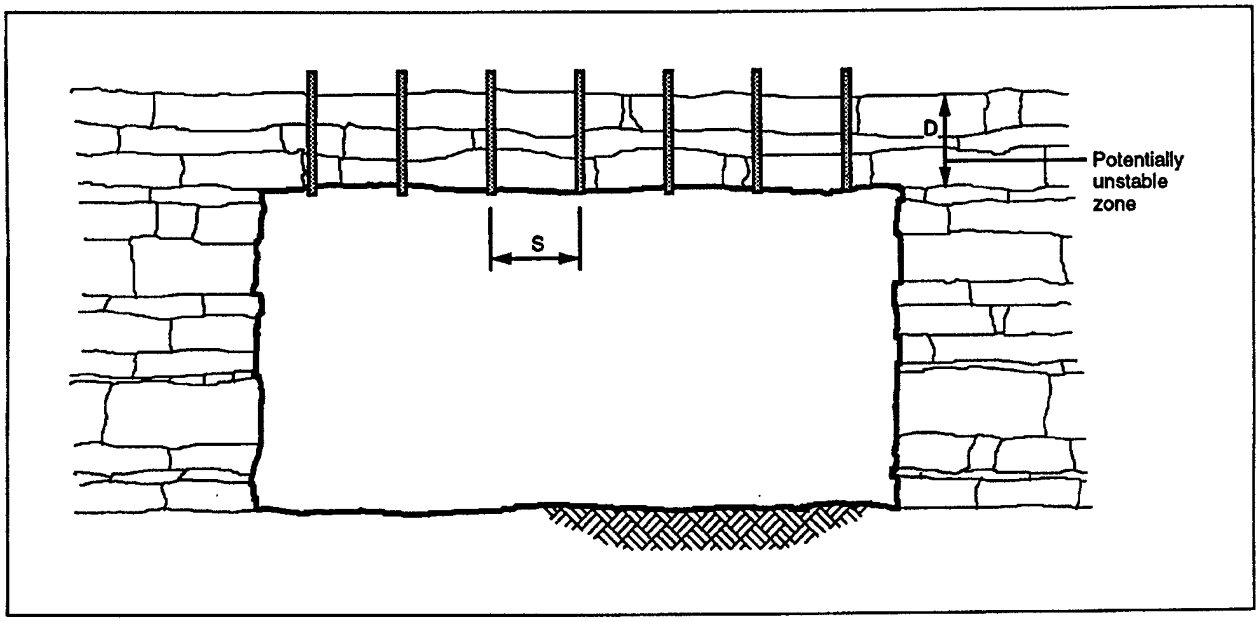


Figure 7-6. Reinforced roof beam

7-4. Design of Steel Ribs and Lattice Girders

In today's tunneling, steel ribs are still used for many purposes. This subsection deals with the selection and design of steel rib supports and lattice girders.

a. Use of steel ribs and lattice girders.

(1) Steel ribs are usually made of straight or bent I-beams or H-beams, bolted together to form a circular or pitched arch with straight, vertical side supports (legs), or a true horseshoe shape with curved legs, sometimes with a straight or curved horizontal invert strut. Full-circle steel sets are also common. Structural shapes other than I- or H-beams have also been used.

(2) Steel sets are most often used as ground support near tunnel portals and at intersections, for TBM starter tunnels, and in poor ground in blasted tunnels. Steel sets are also used in TBM tunnels in poor ground when a reaction platform for propulsion is required. The traditional blocking consists of timber blocks and wedges, tightly installed between the sets and the rock, with an attempt to prestress the set. Timbers not essential for ground support are generally removed before placing a final, cast-in-place concrete lining. Recently, blocking made of concrete or steel is often specified. This method is more difficult to work with, and a more flexible method consists of using special bags pumped full of concrete. These bags will accommodate themselves to the shape of the rock as excavated and form a firm contact with the rock.

(3) Shotcrete is also used as blocking material. When well placed, shotcrete fills the space between the steel rib and the rock and is thus superior to other methods of blocking by providing for a uniform interaction between the ground and the support. Care must be exercised to fill all the voids behind each rib.

(4) Lattice girders offer similar moment capacity at a lower weight than comparable steel ribs. They are easier to handle and erect. Their open lattice permits shotcrete to be placed with little or no voids in the shadows behind the steel structure, thus forming a composite structure. They can also be used together with dowels, spiling, and wire mesh, and (see Figure 5-19) as the final lining.

b. Design of blocked ribs.

(1) The still-popular classical text provided in Proctor and White (1946) is the best guide to the design of steel ribs installed with blocking. The designer is referred to this text for details of design and several design charts and to the available commercial literature for the design of connections and other details. The basic theory behind the classical method of rib design is that the flexibility of the steel rib/timber blocking system permits essentially complete load redistribution. Vertical loads transferred

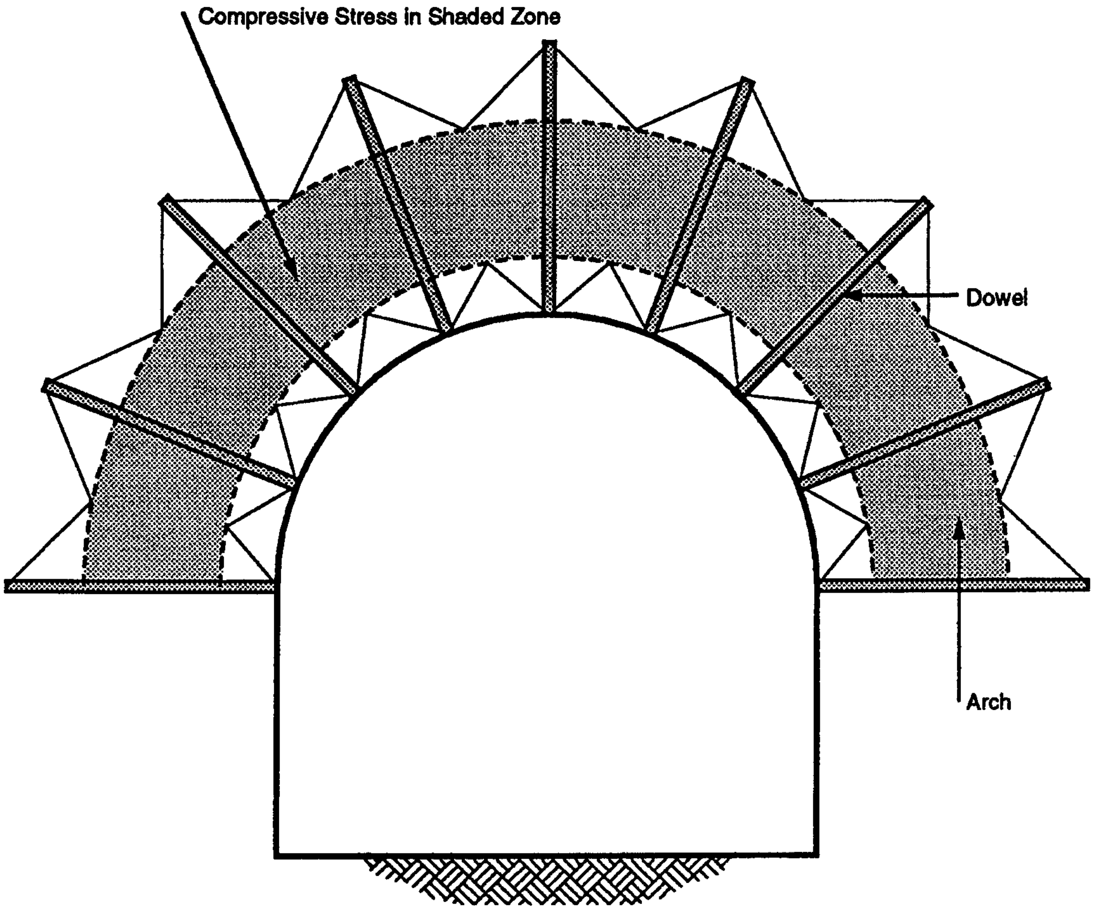


Figure 7-7. Reinforced roof arch

through the blocking cause a deformation sufficient to generate reactions along the sides, such that loads around the arch become essentially uniform. Loads at an angle with vertical have the same effect. Thus, the combined loads result in a uniform thrust in the rib (T), and the maximum moment occurs at blocking points and at points in the middle between blocking points. If the rib was assumed to be pinned at the blocking points, the moment would be equal to the thrust multiplied by the rise of the arc (h) between the blocking points ($M_t = Th$). In fact, the rib is continuous, and there is a moment (M_b) at the blocking points. The maximum moment, then, is $M_m = M_t - M_b$.

(2) If the arch is continuous, fixed at both ends, and bears against equally spaced blocking points, then the maximum moment occurs at blocking points and is approximately $M_{max} = M_b = 0.67\ M_t = 0.67\ Th$. If the arch is hinged at both ends, the maximum moment is 0.86 Th.

(3) When the arch is fixed at the top of a straight leg, the moment in the leg is 0.67 Th, reducing to zero at the bottom, assumed as a hinge. When there are significant side pressures on the legs, the leg moments become larger, the legs must be prevented from kicking in, and arched (horseshoe) legs are often used, together with invert struts.

Table 7-9
Empirical Design Recommendations

Parameter		*Empirical Rule*
Minimum length and maximum spacing		
Minimum length		Greatest of
	(a)	2 x bolt spacing
	(b)	3 x thickness of critical and potentially unstable rock blocks (Note 1)
	(c)	For elements above the springline: spans <6 m: 0.5 x span spans between 18 and 30 m: 0.25 x span
	(d)	For elements below the springline: height <18 m: as (c) above height >18 m: 0.2 x height
Maximum spacing		Least of:
	(a)	0.5 x bolt length
	(b)	1.5 x width of critical and potentially unstable rock blocks (Note 1)
	(c)	2.0 m (Note 2)
Minimum spacing		0.9 to 1.2 m
Minimum average confining pressure		
Minimum average confining pressure at yield point of elements (Note 3)		Greatest of
	(a)	Above springline: *either* pressure = vertical rock load of 0.2 x opening width or 40 kN/m^2
	(b)	Below springline: *either* pressure = vertical rock load of 0.1 x opening height of 40 kNm^2
	(c)	At intersections: 2 x confining pressure determined above (Note 4)

Notes:
1. Where joint spacing is close and span relatively large, the superposition of two reinforcement patterns may be appropriate (e.g., long heavy elements on wide centers to support the span, and shorter, lighter bolts on closer centers to stabilize the surface against raveling).
2. Greater spacing than 2.0 m makes attachment of surface support elements (e.g., weldmesh or chain-link mesh) difficult.
3. Assuming the elements behave in a ductile manner.
4. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections, and rock blocks are free to move toward both openings.

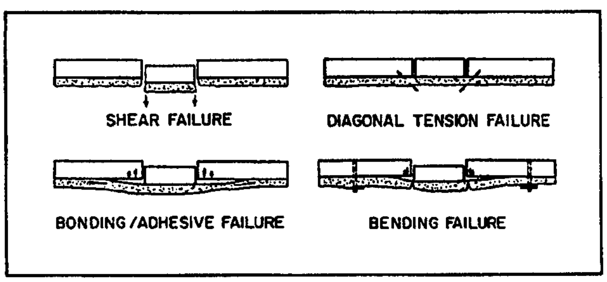


Figure 7-8. Shotcrete failure modes

With very large side pressure, such as in squeezing ground, a full circular shape is used.

c. Lattice girders with continuous blocking.

(1) The theory for blocked arches works adequately for curved structural elements if the blocking is able to deform in response to applied loads, provided the arch transmits a thrust and moment to the end points of the arch. With continuous blocking by shotcrete, however, the

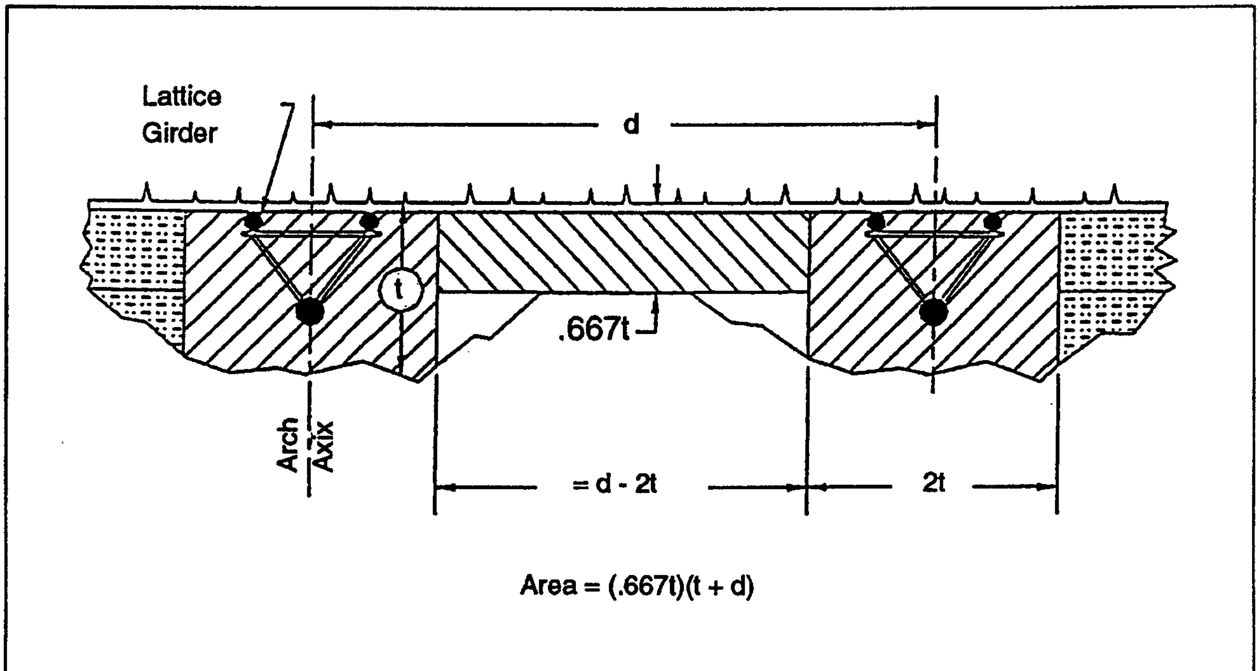


Figure 7-9. Estimation of cross section for shotcrete-encased lattice girders

blocking does not yield significantly once it has set and load redistribution is a function of excavation and installation sequences. Moments in the composite structure should preferably be estimated using one of the methods described in Chapter 9. To estimate moments for sequential excavation and support, where the ground support for a tunnel station may be constructed in stages, finite element or finite difference methods are preferred. These analyses should ideally incorporate at least the following features:

- Unloading of the rock due to excavation.

- Application of ground support.
  - First shotcrete application.
  - Lattice girder installation.
  - Subsequent shotcrete application.
  - Other ground support (dowels, etc.) as applicable.

- Increase in shotcrete modulus with time as it cures.

- Repeat for all partial face excavation sequences until lining closure is achieved.

(2) These types of analysis only yield approximate results. However, they are useful to study variations in construction sequences, locations of maximum moments and thrusts, and effects of variations of material properties and in situ stress.

(3) Stresses in composite lattice girder and shotcrete linings can be analyzed in a manner similar to reinforced concrete subjected to thrust and bending (see Chapter 9). Figure 7-9 shows an approximation of the typical application of lattice girders and shotcrete. The moment capacity analysis should be performed using the applicable shotcrete strength at the time considered in the analysis.

Chapter 8
Geomechanical Analyses

Understanding rock mass response to tunnel and shaft construction is necessary to assess opening stability and opening support requirements. Several approaches of varying complexity have been developed to help the designer understand rock mass response. The methods cannot consider all aspects of rock behavior, but are useful in quantifying rock response and providing guidance in support design.

8-1. General Concepts

a. Stress/strain relationships.

(1) *Elastic parameters.*

(a) Elasticity is the simplest and most frequently applied theory relating stress and strain in a material. An elastic material is one in which all strain is instantaneously and totally recoverable on the removal of the stress. The theory of elasticity idealizes a material as a linear elastic, isotropic, homogeneous material.

(b) The stress/strain relationship for rock can sometimes be idealized in terms of a linear elastic isotropic material. In three dimensions, for an isotropic homogeneous elastic material subject to a normal stress σ_x in the x direction, the strains in the x, y, and z directions are:

$$\varepsilon_x = \sigma_x/E \qquad \varepsilon_y = \varepsilon_z = -\nu \cdot \sigma_x/E$$

where

ε_x = applied stress in x-direction

ν = Poisson's Ratio

E = modulus of elasticity

Since the principle of superposition applies, the stress/strain relationships in three dimensions are:

$$\varepsilon_x = (\sigma_x - \nu(\sigma_y + \sigma_z))/E$$

$$\varepsilon_y = (\sigma_y - \nu(\sigma_z + \sigma_x))/E$$

$$\varepsilon_z = (\sigma_y - \nu(\sigma_y + \sigma_x))/E$$

(c) For a competent rock that is not linear elastic, the stress/strain relationship can be generalized in the form of a curve with an increasing slope at low stress levels (related to closing of microcracks), an approximately linear zone of maximum slope over its midportion, and a curve of decreasing slope at stress levels approaching failure. In order to apply elastic theory to such rocks, it is necessary to define an approximate modulus of elasticity. The different methods available for defining this modulus of elasticity are as follows:

- Tangent modulus (E_T) to a particular point on the curve, i.e., at a stress level that is some fixed percentage (usually 50 percent) of the maximum strength.

- Average slope of the more-or-less straight line portion of the stress/strain curve.

- Secant modulus (E_s) usually from zero to some fixed percentage of maximum strength.

(d) Since the value of Poisson's Ratio is greatly affected by nonlinearities in the axial and lateral stress-strain curves at low stress levels, ASTM suggests that the Poisson's Ratio is calculated from the equation:

$$\nu = \textit{slope of axial curve/slope of lateral curve}$$

(e) For most rocks, Poisson's Ratio lies between 0.15 and 0.30. Generally, unless other information is available, Poisson's Ratio can be assumed as 0.25. The modulus of elasticity varies over a wide range. For crude estimating purposes, the modulus of elasticity is about 350 times the uniaxial compressive strength of a rock (Judd and Huber 1961).

(f) Establishing values for elastic parameters that apply in the field takes judgment and should be made on a case-by-case basis. For a strong but highly jointed rock mass, a reduction in the value of E from the laboratory values of an order of magnitude may be in order. On the other hand, when testing very weak rocks (uniaxial compressive strength less than 3.5 MPa (500 psi)), sample disturbance caused by the removal of the rock sample from the ground may introduce defects that result in reduced values for the laboratory-determined modulus. For critical projects it is advisable to use field tests to determine the in situ deformability of rock.

(2) *Nonelastic parameters.* Many rocks can be characterized as elastic without materially compromising the

analysis of their performance. Where the stresses are sufficiently large that a failure zone develops around the tunnel, elastoplastic analyses are available for analyzing the stresses and strains. However, for some rocks such as potash, halite, and shales, time-dependent or creep movements may be significant and must be taken into account when predicting performance. Chabannes (1982) has established the time-dependent closure based on a steady-state creep law. Lo and Yuen (1981) have used rheological models to develop a design methodology for liner design that has been applied to shales. Time-dependent relationships are difficult to characterize because of the difficulty selecting rock strength parameters that accurately model the rock mass.

(3) *Rock strength.* Rock material is generally strong in compression where shear failure can occur and weak in tension. Failure can take the form of fracture, in which the material disintegrates at a certain stress, or deformation beyond some specific strain level. Rocks exhibit a brittle-type behavior when unconfined, but become more plastic as the level of confinement increases. Conditions in the field are primarily compressive and vary from unconfined near the tunnel walls to confined some distance from the tunnel. The strength of a rock is affected not only by factors that relate to its physical and chemical composition such as its mineralogy, porosity, cementation, degree of alteration or weathering, and water content, but also by the method of testing, including such factors as sample size, geometry, test procedure, and loading rate.

(4) *Uniaxial compressive strength.*

(a) The uniaxial or unconfined compressive strength is the geotechnical parameter most often quoted to characterize the mechanical behavior of rock. It can be misleading since field performance often depends on more than just the strength of an intact sample, and this value is subject to a number of test-related factors that can significantly affect its value. These factors include specimen size and shape, moisture content, and other factors. Uniaxial compressive strength usually should not be considered a failure criterion but rather an index that gives guidance on strength characteristics. It is most useful as a means for comparing rocks and classifying their likely behavior.

(b) The compressive strength of a rock material is size dependent, with strength increasing as specimen size decreases. It is useful to adjust the compressive strength values to take into account the size effect. An approximate relationship between uniaxial compressive strength and specimen diameter that allows comparison between samples is as follows:

$$\sigma_c = \sigma_{c50}\ (50/d)^{0.18}$$

where

σ_{c50} = compressive strength for a 50-mm- (2-in.-) diam sample

d = sample diameter (Hoek and Brown 1980)

(c) The compressive strength of a rock material often decreases when the rock is immersed in water. The reduced stresses may be due to dissolution of the cementation binding the rock matrix or to the development of water pressures in the interconnected pore space.

(5) *Tensile strength.* For underground stability, the tensile strength is not as significant a parameter as the compressive strength for rocks. Generally, tensile rock strength is low enough that when rock is in tension, it splits and the tensile stresses are relieved. As a rule of thumb, the tensile strength of rock material is often taken as one-tenth to one-twelfth of the uniaxial compressive strength of the intact rock. In jointed rocks, the jointing may very well eliminate the tensile strength of the rock mass, in which case the in situ rock should be considered as having zero tensile strength. Values of tensile strength and other geotechnical parameters of some intact rocks are given in Table 8-1.

(6) *Mohr-Coulomb failure criterion.*

(a) The Mohr-Coulomb failure criterion is most often applied to rock in the triaxial stress state. This criterion is based on (1) rock failure occurring once the shear stress on any plane reaches the shear strength of the material, (2) the shear strength along any plane being a function of the normal stress G_n on that plane, and (3) the shear strength being independent of the intermediate principal stress. The general form of the normal stress versus shear stress plot is shown in Figure 8-1. As an approximation over limited ranges of normal stress, the shear stress is defined as a linear relationship of the normal stress as follows:

$$\tau = c + \sigma_n \times \tan \phi$$

where

τ = shear strength

σ_n = applied normal stress

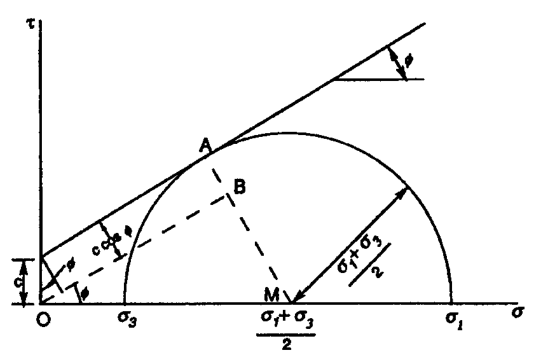
Table 8-1
Geotechnical Parameters of Some Intact Rocks (after Lama and Vutukuri 1978)

Rock Type	Location	Density Mg/m³	Young's Modulus, GPa	Uniaxial Compressive Strength, MPa	Tensile Strength MPa
Amphibolite	California	2.94	92.4	278	22.8
Andesite	Nevada	2.37	37.0	103	7.2
Basalt	Michigan	2.70	41.0	120	14.6
Basalt	Colorado	2.62	32.4	58	3.2
Basalt	Nevada	2.83	33.9	148	18.1
Conglomerate	Utah	2.54	14.1	88	3.0
Diabase	New York	2.94	95.8	321	55.1
Diorite	Arizona	2.71	46.9	119	8.2
Dolomite	Illinois	2.58	51.0	90	3.0
Gabbro	New York	3.03	55.3	186	13.8
Gneiss	Idaho	2.79	53.6	162	6.9
Gneiss	New Jersey	2.71	55.2	223	15.5
Granite	Georgia	2.64	39.0	193	2.8
Granite	Maryland	2.65	25.4	251	20.7
Granite	Colorado	2.64	70.6	226	11.9
Graywacke	Alaska	2.77	68.4	221	5.5
Gypsum	Canada	-	-	22	2.4
Limestone	Germany	2.62	63.8	64	4.0
Limestone	Indiana	2.30	27.0	53	4.1
Marble	New York	2.72	54.0	127	11.7
Marble	Tennessee	2.70	48.3	106	6.5
Phyllite	Michigan	3.24	76.5	126	22.8
Quartzite	Minnesota	2.75	84.8	629	23.4
Quartzite	Utah	2.55	22.1	148	3.5
Salt	Canada	2.20	4.6	36	2.5
Sandstone	Alaska	2.89	10.5	39	5.2
Sandstone	Utah	2.20	21.4	107	11.0
Schist	Colorado	2.47	9.0	15	-
Schist	Alaska	2.89	39.3	130	5.5
Shale	Utah	2.81	58.2	216	17.2
Shale	Pennsylvania	2.72	31.2	101	1.4
Siltstone	Pennsylvania	2.76	30.6	113	2.8
Slate	Michigan	2.93	75.9	180	25.5
Tuff	Nevada	2.39	3.7	11	1.2
Tuff	Japan	1.91	76.0	36	4.3

c = cohesion of the rock

ϕ = angle of internal friction

(b) Generally, the shear strength in the laboratory is determined from testing intact rock samples in compression. The value obtained in this way does not take into account the joints and other discontinuities that materially influence the strength behavior of the rock mass.



Linear Failure Envelope Defined by the Equation $\tau = c + \sigma_n \tan \phi$

The stress at a point in a state of incipient failure is represented by the circle through the points representing the minimum principal stress σ_3 and the maximum principal stress σ_1 at that point.

c = cohesion or the rock
ϕ = angle of internal friction of the rock

Figure 8-1. Mohr-Coulomb failure criterion

(7) *Hoek-Brown failure criterion.*

(a) To overcome the difficulties in applying the Mohr-Coulomb theory to rocks, i.e., the nonlinearity of the actual failure envelope and the influence of discontinuities in the rock mass, Hoek and Brown (1980) developed an empirical failure criterion. The Hoek-Brown failure criterion is based on a combination of field, laboratory, and theoretical considerations, as well as experience. It sets out to describe the response of an intact sample to the full range of stress conditions likely to be encountered. These conditions range from uniaxial tensile stress to triaxial compressive stress. It provides the capability to include the influence of several sets of discontinuities. This behavior may be highly anisotropic.

(b) The Hoek-Brown failure criterion is as follows:

$$\sigma_1 = \sigma_3 + \sqrt{m\,\sigma_c\,\sigma_3 + s\,\sigma_c^2}$$

where

σ_1 = major principal stress at failure

σ_3 = minor principal stress at failure

σ_c = uniaxial compressive strength of the intact rock material (given by $\sigma_3 = 0$ and s = 1)

m and *s* are constants that depend on the properties of the rock and the extent to which it has been broken before being subjected to the stresses σ_1 and σ_3.

(c) In terms of shear and normal stresses, this relationship can be expressed as:

$$\tau = (\sigma \times \sigma_3)\sqrt{1 \times m\,\sigma_c/4\,\tau_m}$$

where

$$\tau_m = 0.5\,(\sigma_1 - \sigma_3)$$

(d) Hoek and Brown (1988) have developed estimates for the strengths of rock masses based on experience with numerous projects. The estimates that cover a wide range of rock mass conditions are given in Table 8-2.

b. In situ stress conditions. The virgin or undisturbed in situ stresses are the natural stresses that exist in the ground prior to any excavation. Their magnitudes and orientation are determined by the weight of the overlying strata and the geological history of the rock mass. The principal stress directions are often vertical and horizontal. They are likely to be similar in orientation and relative magnitude to those that caused the most recent deformations. Some of the simplest clues to stress orientation can be estimated from a knowledge of a region's structural geology and its recent geologic history. Knowledge of undisturbed stresses is important. They determine the boundary conditions for stress analyses and affect stresses and deformations that develop when an opening is created. Quantitative information from stress analyses requires that the boundary conditions are known. Uncertainties are introduced into the analyses by limited knowledge of in situ stresses. Although initial estimates can be made based on simple guidelines, field measurements of in situ stresses are the only true guide for critical structures.

(1) *In situ vertical stress.* For a geologically undisturbed rock mass, gravity provides the vertical component of the rock stresses. In a homogeneous rock mass, when the rock density γ is constant, the vertical stress is the pressure exerted by the mass of a column of rock acting over level. The vertical stress due to the overlying rock is then:

$$\sigma_z = \gamma h$$

Table 8-2
Approximate Relationship Between Rock Mass Quality and Material Constants Applicable to Underground Works

	Carbonate Rocks with Well Developed Crystal Cleavage *dolomite, limestone, and marble*	**Lithified Agrillaceous Rocks** *mudstone, siltstone, shale, and slate (normal to cleavage)*	**Arenaceous Rocks with Strong Crystals and Poorly Developed Crystal Cleavage** *sandstone and quartzite*	**Fine-Grained Polymineralic Igneous Crystalline Rocks** *andesite, dolerite, diabase, and rhyolite*	**Coarse-Grained Polymineralic Igneous and Metamorphic Crystalline Rocks** *amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*
Intact Rock Samples *Laboratory specimens free from discontinuities* RMR = 100, Q = 100	m = 7.00 s = 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
Very Good Quality Rock Mass *Tightly interlocking undisturbed rock with unweatherd joints at 1 to 3 m* RMR = 85, Q = 100	m = 4.10 s = 0.189	5.85 0.189	8.78 0.189	9.95 0.189	14.63 0.189
Good Quality Rock Mass *Several sets of moderately weathered joints spaced at 0.3 to 1 m* RMR = 65, Q = 10	m = 2.006 s = 0.0205	2.865 0.0205	4.298 0.0205	4.871 0.0205	7.163 0.0205
Fair Quality Rock Mass *Several sets of moderately weathered joints spaced at 0.3 to 1 m* RMR = 44, Q = 1	m = 0.947 s = 0.00198	1.353 0.00198	2.030 0.00198	2.301 0.00198	3.383 0.00198
Poor Quality Rock Mass *Numerous weathered joints at 30-500 mm, some gouge; clean compacted waste rock* RMR = 23, Q = 0.1	m = 0.447 s = 0.00019	0.639 0.00019	0.959 0.00019	1.087 0.00019	1.598 0.00019
Very Poor Quality Rock Mass *Numerous heavily weathered joints spaced < 50 mm with gouge; waste rock with fines* RMR = 3, Q = 0.01	m = 0.219 s = 0.00002	0.313 0.00002	0.469 0.00002	0.532 0.00002	0.782 0.00002

Empirical Failure Criterion:

$$\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$$

σ_1' = major principal effective stress
σ_3' = minor principal effective stress
σ_c' = uniaxial compressive strength of intact rock, and *m* and *s* are impirical constants
CSIR rating: RMR
NGI rating: Q

where γ represents the density that is the unit weight of the rock and generally lies between 20 and 30 kN/m^3.

(2) *In situ horizontal stress*. The horizontal in situ stresses also depend on the depth below surface. They are generally defined in terms of the vertical stress as follows:

$$K_o = \sigma_h / \sigma_v$$

where K_0 represents the lateral rock stress ratio. Since there are three principal stress directions, there will be two horizontal principal stresses. In an undisturbed rock mass, the two horizontal principal stresses may be equal, but generally the effects of material anisotropy and the geologic history of the rock mass ensure that they are not. The value of K_O is difficult to estimate without field measurements. However, some conditions exist for which reasonable estimates can be made. Guidelines for these estimates are as follows:

(a) For weak rocks unable to support large deviatoric stress differences, the lateral and vertical stresses tend to equalize over geologic time. This is called Heim's Rule.

$$\sigma_x = \sigma_y \cong \sigma_z$$

Lithostatic stress occurs when the stress components at a point are equal in all directions and their magnitude is due to the weight of overburden. A lithostatic stress state is widely used in weak geologically undisturbed sediments exhibiting plastic or visco-plastic behavior, such as coal measures, shales, mudstones, and evaporites. It also gives reasonable estimates of horizontal stresses at depths in excess of 1 km.

(b) A lower limiting value of K_O derives from the assumption that the rock behaves elastically but is constrained from deforming horizontally. This applies to sedimentary rocks in geologically undisturbed regions where the strata behave linearly elastically and are built up in horizontal layers such that the horizontal dimensions are unchanged. For this case, the lateral stresses σ_x and σ_y are equal and are given by:

$$\sigma_x = \sigma_y = \gamma\ h\ \nu/(1-\nu)$$

Since Poisson's Ratio for most rocks lies between 0.15 and 0.35, the value of K_O should lie between about 0.2 and 0.55. For a typical rock with a Poisson's Ratio of 0.25, the undisturbed lateral stresses would be 0.33 times the vertical stress. This approach provides a lower bound estimate that applies under appropriate geological conditions.

(c) Amadei, Swolfs, and Savage (1988) have shown that the inclusion of anisotropy broadens the range of permissible values of gravity-induced horizontal stresses in rock masses. For some ranges of anisotropic rock properties, gravity-induced horizontal stresses exceed the vertical stress. Amadei, Swolfs, and Savage have shown that this can be extended to stratified or jointed rock masses.

(d) Residual stresses are the stresses remaining in rock masses after their causes have been removed. During a previous history of a rock mass, it may have been subjected to higher stresses than it is subjected to at the present time. On removal of the load causing the higher stresses, the relaxation of the rock is resisted by the interlocking mineral grains, the shear stresses along fractures, and cementation between particles.

(e) Tectonic stresses are due to previous and present-day straining of the earth's crust. They may arise from regional uplift, downwarping, faulting, folding, and surface irregularities. Tectonic stresses may be active or remnant, depending on whether they are due to present or partially relieved past tectonic events, respectively. The superposition of these tectonic stresses on the gravity-induced stress field can result in substantial changes in both the direction and the magnitude of the resultant primitive stresses. Tectonic and residual stresses are difficult to predict without actual measurement. The evaluation of the in situ state of stress requires knowledge of the regional geology, stress measurements, and observations of the effects of natural stresses on existing structures in rock.

(f) The state of stress at the bottom of a V-shaped valley is influenced by the geometry of both the valley and the hills—the topography.

(3) *In situ stress measurements*.

(a) During the past 20 years, methods for measuring in situ stresses have been developed and a database established. Based on a survey of published results, Hoek and Brown (1980) have compiled a survey of published data that is summarized in Figure 8-2. The data confirm that the vertical stresses measured in the field reasonably agree with simple predictions using the overlying weight of rock.

(b) Horizontal in situ stress rarely show magnitudes as low as the limiting values predicted by elastic theory. The measurements often indicate high stresses that are

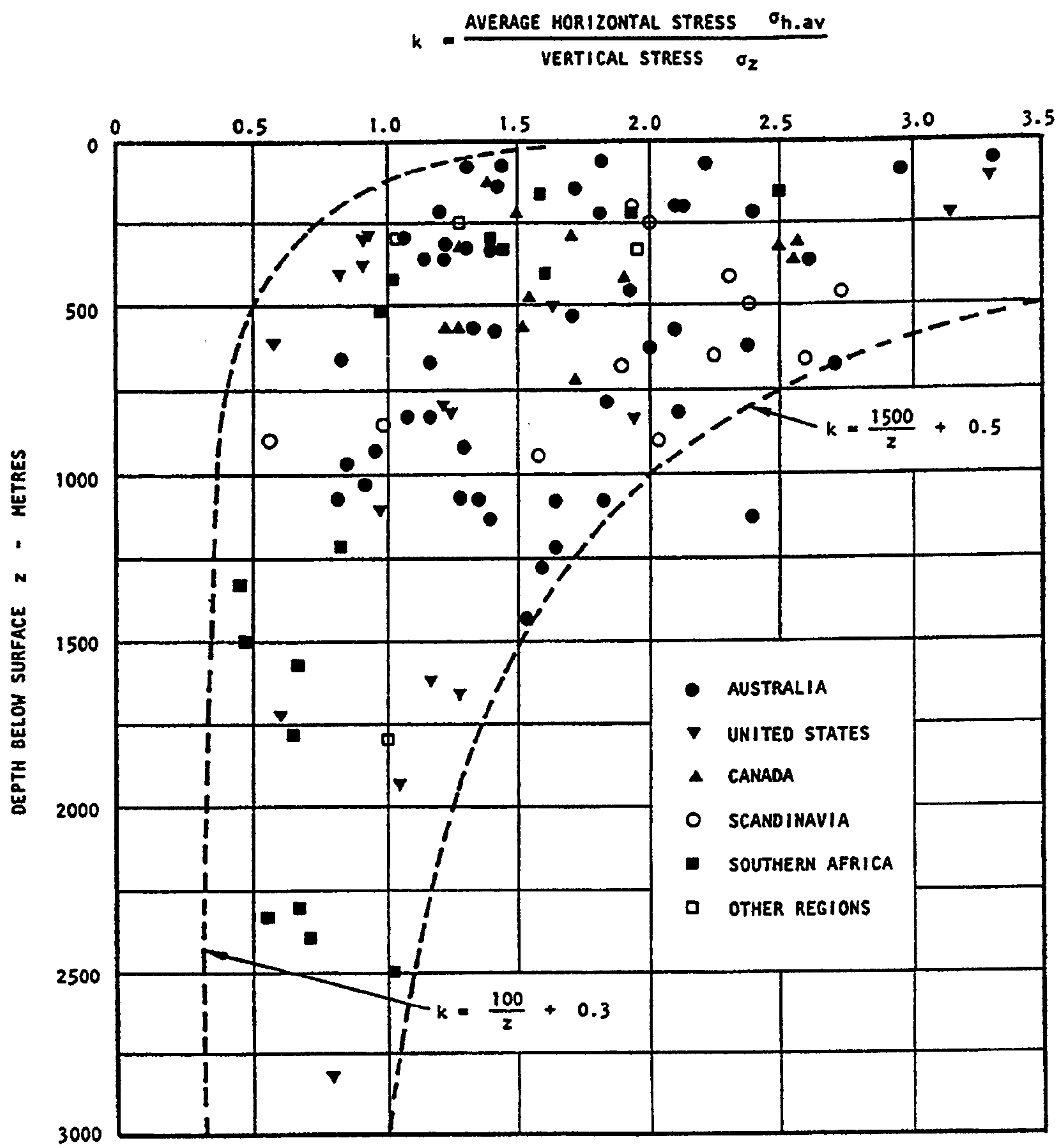


Figure 8-2. Variation of ratio of average horizontal stress to vertical stress with depth below surface

attributed to denudation, tectonics, or surface topography. The horizontal stresses vary considerably and depend on geologic history. At shallow depths, there may be a wide variation in values since the strain changes being measured are often close to the limit of the accuracy of the measuring tools.

8-2. Convergence-Confinement Method

a. The convergence-confinement method combines concepts of ground relaxation and support stiffness to determine the interaction between ground and ground support. As an example, Figure 8-3 illustrates the concept of rock-support interaction in a circular tunnel excavated by a TBM. The ground relaxation curve shown represents poor rock that requires support to prevent instability or collapse. The stages described in Figure 8-3 are outlined below:

b. An early installation of the ground support (Point D_1) leads to excessive buildup of load in the support. In a yielding support system, the support will yield (without collapsing) to reach equilibrium Point E_1. A delayed installation of the support (Point D_2) leads to excessive tunnel deformation and support collapse (Point E_2). The designer can optimize support installation to allow for acceptable displacements in the tunnel and loads in the support.

c. The convergence-confinement method is not limited to the construction of rock-support interaction curves. The method is a powerful conceptual tool that provides the designer with a framework for understanding support behavior in tunnels and shafts. The closed-form solutions (Section 8-3) or continuum analyses (Section 8-4) are convergence-confinement methods as they model the rock-structure interaction. The ground relaxation/ interaction curve can also be defined by in situ measurements.

8-3. Stress Analysis

The construction of an underground structure within a rock mass differs from most other building activities. Generally, an aboveground structure is built in an unstressed environment with loads applied as the structure is constructed and becomes operational. For an underground structure, the excavation creates space within a stressed environment. Stress analyses provide insight into the changes in preexisting stress equilibrium caused by an opening. It interprets the performance of an opening in terms of stress concentrations and associated deformations and serves as a rational basis for establishing the performance of requirements for design. The properties of the rock mass are complex, and no single theory is available to explain rock mass behavior. However, the theories of elasticity and plasticity provide results that have relevance to the stress distributions induced about openings and provide a first step to estimating the distribution of stresses around openings. Prior to excavation, the in situ stresses in the rock mass are in equilibrium. Once the excavation is made, the stresses in the vicinity of the opening are redistributed and stress concentrations develop. The redistributed stresses can overstress parts of the rock mass and make it yield. The initial stress conditions in the rock, its geologic structure and failure strength, the method of excavation, the installed support, and the shape of the opening are the main factors that govern stress redistribution about an opening.

a. Excavation configuration and in situ stress state. The excavation shape and the in situ stresses affect the stress distribution about an opening. Since stress concentrations are often critical in the roof and sidewalls of excavations, Hoek and Brown (1980) have determined the tangential stresses on the excavation surface at the crown and in the sidewall for different-shaped openings for a range of in situ stress ratios. They are given in Figure 8-4. These are not necessarily the maximum stresses developing about the opening. Maximum stresses occur at the corners where they can cause localized instabilities such as spalling.

b. Porewater pressures. Stress analysis within the rock mass for tunnelling has been traditionally carried out in terms of total stresses with little consideration given to pore pressures. However, as design approaches for weak permeable rocks are improved, design approaches in terms of effective stress analyses are being developed (Fernandez and Alvarez 1994; Hashash and Cook 1994, see Section 8-4).

c. Circular opening in elastic material. The elastic solution for a deep circular tunnel provides insight into the stresses and displacements induced by the excavation. The tunnel is regarded as "deep" if the free surface does not affect the stresses and displacements around the opening. The problem is considered a plane strain problem and the rock assumed to be isotropic, homogeneous, and linearly elastic. Kirsch's solution (Terzaghi and Richart 1952) disregards body forces and the influence of the boundary at the ground surface. Mindlin's comprehensive solution (1939), which considers the boundary and takes gravity into account, shows that the approximation gives very good agreement for the stresses for depths greater than about four tunnel diameters. Absolute values of stress and

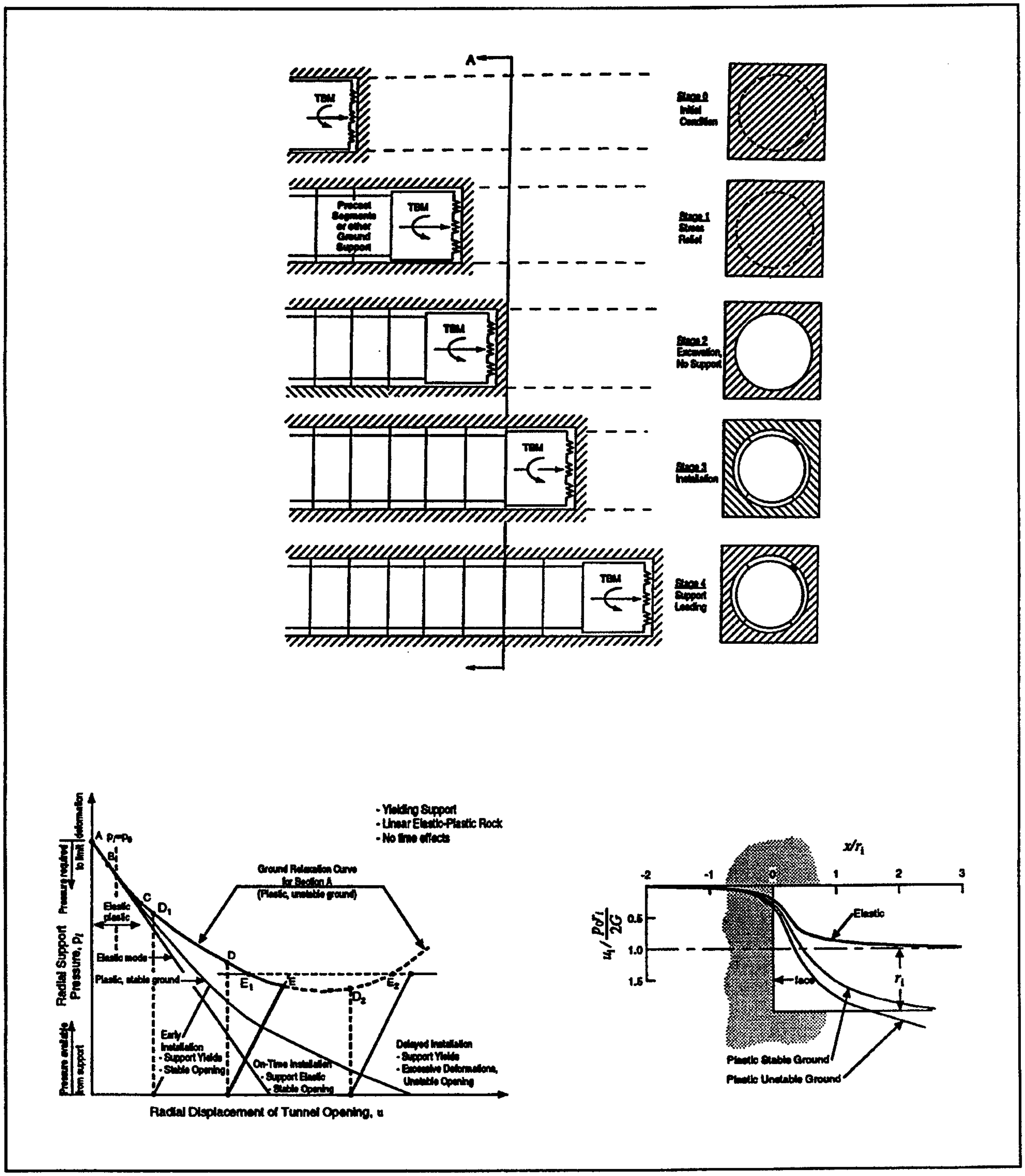
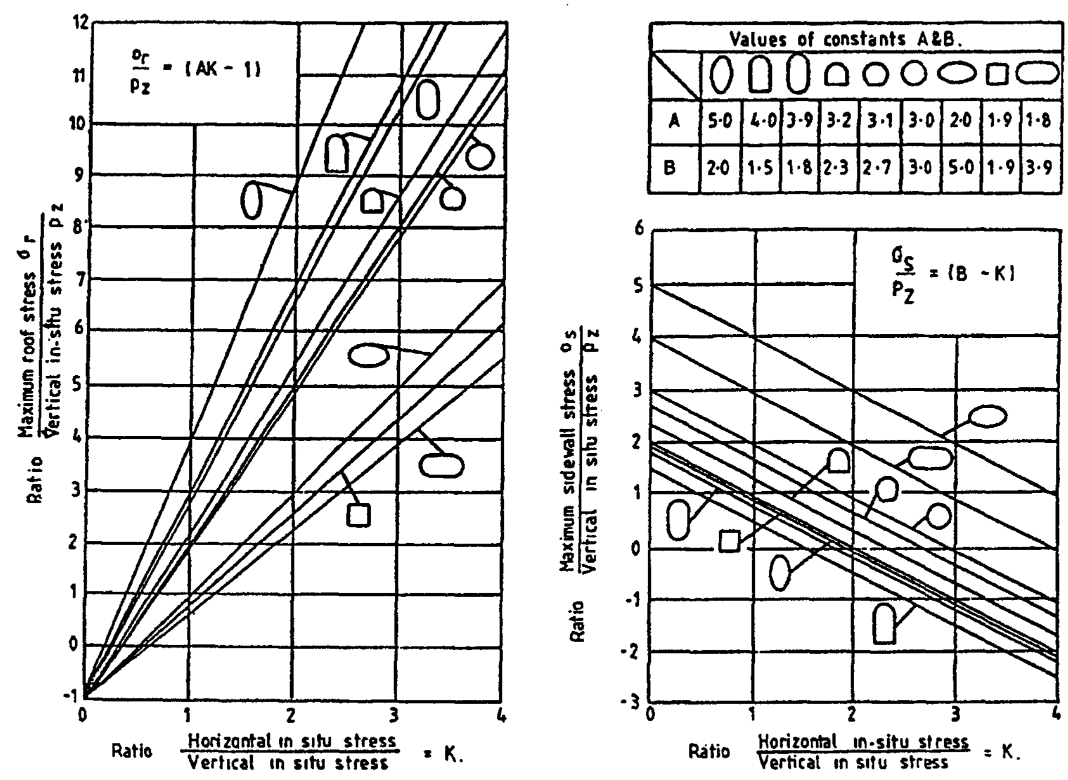


Figure 8-3. Rock-support interaction



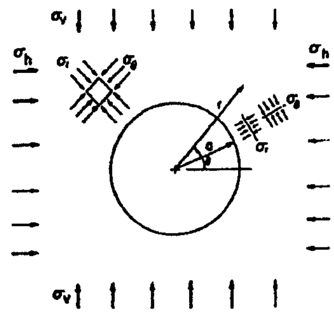
(After Hoek & Brown, 1980)

Figure 8-4. Stresses predicted by elastic analysis

deformation are the same regardless of the sequence of application of loading and excavation; however, relative displacements experienced when the tunnel is driven can only be determined theoretically. Pender (1980) has presented comprehensive solutions for the linear elastic plane strain problem that are summarized in Box 8-1. The simplicity of the elastic solution for the stresses and displacements about a circular opening provides insight into the significance of various parameters and can be used to understand the magnitude of the stresses and deformations induced about an opening.

d. Plastic/yield model. The creation of an underground excavation disturbs the stress field. In the case of weak or even competent rocks subject to high stresses, induced stresses can exceed the strength of the rock leading to its failure. Failure takes the form of gradual closure of the excavation, localized spalling, roof falls, slabbing of side walls, or, in extreme cases, rock bursts. In cases where the violent release of energy is not a factor, this leads to the development of a fractured zone about an excavation that will require stabilization. In strong rocks where brittle or strain softening behavior occurs, strata can be supported relatively easily by the mobilization of the residual strength of the deformed strata by low support pressures. In weaker rocks subject to high stresses where ductile or strain-hardening behavior occurs, possibly over a period of time, much higher restraint is required to support strata; as part of the development of a yield zone, substantial plastic or time-dependent deformations may occur. To estimate these effects, stresses and deformations are

Box 8-1. Stresses Around a Circular Opening in a Biaxial Stress Field



Notation used to describe stresses around a circular opening in a biaxial stress field

a	radius of tunnel shaft
r	radial distance to any point
θ	angular distance to any point
σ_h, σ_v	original (pre-tunneling) stress field at the tunnel level
σ_θ, σ_r	final (post tunneling) radial and tangential stresses around the tunnel
E	is Young's Modulus of the rock
ν	is the Poisson's Ratio
u_a	is the radial displacement at radius a
v_a	is the tangential displacement at radius a

The stresses are:

radial stress $\sigma_r = 0.5(\sigma_v + \sigma_h)(1 - a^2/r^2) + 0.5\,(\sigma_v - \sigma_h)(1 + 3a^4/r^4 - 4a^2/r^2) \cos 2\theta$

circumferential stress $\sigma_\theta = 0.5(\sigma_v + \sigma_h)(1 + a^2/r^2) - 0.5\,(\sigma_v - \sigma_h)\,(1 + 3a^4/r^4) \cos 2\theta$

shear stress $\tau_{r\theta} = 0.5(\sigma_h - \sigma_v)(1 - 3a^4/r^4 + 2a^2/r^2) \sin 2\theta$

Case 1 Stresses applied at a distant boundary - appropriate for condition where a large surface loading is applied after the tunnel is constructed

The displacements are:

$$Eu = (1-\nu^2)(0.5(\sigma_v + \sigma_h)(r + a^2/r) - 0.5(\sigma_v - \sigma_h)(r - a^4/r^3 + 4a^2/r) \cos 2\theta\} - \nu(1 + \nu)\{0.5(\sigma_v + \sigma_h)(r - a^2/r) - 0.5(\sigma_v - \sigma_h)(r - a^4/r^3) \cos 2\theta\}$$

$$Ev = 0.5(\sigma_v - \sigma_h)\,\{(1 - \nu^2)(r + 2a^2/r + a^4/r^3) + \nu(1+\nu)(r - 2a^2/r + a^4/r^3)\} \sin 2\theta$$

At the tunnel periphery, the displacements are:

$$Eu_a = (1-\nu^2)a\{(\sigma_v + \sigma_h) - 2(\sigma_v - \sigma_h) \cos 2\theta\}$$

$$Ev_a = 2(1-\nu^2)a(\sigma_v - \sigma_h) \sin 2\theta$$

Case 2 Tunnel excavated in a prestressed medium - appropriate for analysis of tunnel excavation

The displacements are:

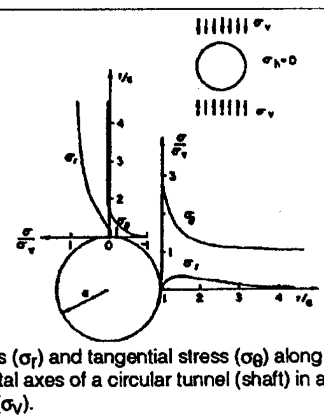
$$Eu = 0.5(1+\nu)\{(\sigma_v + \sigma_h)(a^2/r) - (\sigma_v - \sigma_h)((1-\nu)4a^2/r - a^4/r^3)) \cos 2\theta\}$$

$$Ev = 2(1+\nu)(\sigma_v - \sigma_h)2a^2/r + a^4/r^3)) \sin 2\theta$$

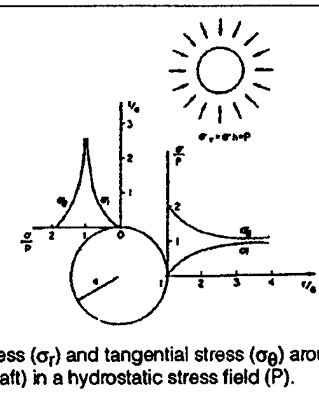
At the tunnel periphery, the displacements are:

$$Eu_a = 0.5(1+\nu)a\{(\sigma_v + \sigma_h) - (3-4\nu)(\sigma_v - \sigma_h) \cos 2\theta\}$$

$$Ev_a = 6(1+\nu)(\sigma_v - \sigma_h) \sin 2\theta$$



Radial stress (σ_r) and tangential stress (σ_θ) along the vertical and horizontal axes of a circular tunnel (shaft) in a uniaxial stress field (σ_v).



Radial stress (σ_r) and tangential stress (σ_θ) around a circular tunnel (shaft) in a hydrostatic stress field (P).

calculated from elasto-plastic analyses. The simplest case is that of a circular tunnel driven in a homogeneous, isotropic, initially elastic rock subject to a hydrostatic stress field. The analysis is axisymmetric. The solution assumes plane strain conditions in the axial direction and that the axial stress remains the principal intermediate stress. As the stresses induced by the opening exceed the yield strength of the rock, a yield zone of radius R, develops about the tunnel while the rock outside the yield zone remains elastic. The analysis is illustrated in Boxes 8-2 through 8-5. The rock tends to expand or dilate as it breaks, and displacements of the tunnel wall will be greater than those predicted by elasticity theory. Support requirements are theoretically related to the displacement of the excavation. Deformations are limited by applying a high support pressure, whereas, support pressures are reduced as deformations take place. These theoretical provisions must be tempered with judgment since excessive deformation can adversely affect stability and lead to increased support requirements that are not predicted by the analyses. The elastoplastic solutions for stress distributions and deformations around circular-cylindrical underground openings are summarized in Boxes 8-2, 8-3, and 8-4. It is assumed that the opening is far enough removed from the ground surface that the stress field may be assumed homogeneous and that a lithostatic stress field exists. Body forces are not considered. The assumption is made that the material is either plastic frictionless ($\phi = 0$) or frictional (c-ϕ).

Box 8-2. Elasto Plastic Solution

Reference: Salencon 1969.

$p_z = \sigma_v = \sigma_n$ $\qquad$ p_i = Internal Pressure

yield condition: $p_z \geq (p_i + c \cos\phi) / 1 - \sin\phi$

radius of yield zone:

$R = a.[(1 - \sin\phi)(p_z + c.\cot\phi) / (p_i + c \cot\phi)]^{1/(Kp-1)}$

where $K_p = (1 + \sin\phi) / (1 - \sin\phi)$

ELASTIC ZONE:

stresses: $\sigma_r = p_z - (p_z - \sigma_{cp}) (R_o/r)^2$

$\sigma_\theta = p_z + (p_z - \sigma_{cp})(R_o/r)^2$

$\sigma_{cp} = p_z (1 - \sin\phi) - c.\cos\phi$ = Radial stress at the Elasto-Plastic interface

deformations: $u_r = (p_z .\sin\phi + c.\cos\phi).(R^2/r) / (2G)$

PLASTIC ZONE:

stresses: $\sigma_r = -c.\cot\phi + (p_i + c.\cot\phi).(r/a)^{Kp-1}$

$\sigma_\theta = -c.\cot\theta + (p_i + c.\cot\phi).K_p.(r/a)^{Kp-1}$

$\sigma_y = (\sigma_r + \sigma_\theta)/2 = c.\cot\phi + (p_i + c.\cot\phi).(1 - \sin\phi)^{-1}.(r/a)^{Kp-1}$

deformations: $u_r = r/(2G).\ \chi$

where $\chi = (2v-1).(p_z + c.\cot\phi) + (1-v).[(K_p^2 - 1)(K_p + K_{ps})] . (p_i + c.\cot\phi). (R/a)^{(Kp-1)}.(R/r)^{(Kps+1)}$

$+ [(1-v).(K_p.K_{ps} + 1)/(K_p + K_{ps}) - v].(p_i + c.\cot\phi).(r/a)^{(Kp-1)}$

and $K_{ps} = (1 + \sin\psi_s) / (1 - \sin\psi s)$ and $G = E/2(1+v)$

Box 8-3. Elasto Plastic Particular Solutions

Particular Solutions to Elastoplastic Problem - c-ϕ Material - Dilation Angle stresses in the elastic and plastic zones are the same as given in Box 8-2.

CASE 1: $\Psi = \phi$, Associated flow rule, $K_p = K_{ps}$
deformations: $u_r = r/(2G).\chi$
where $\chi = (2v-1).(p_z + c.\cot\phi) + (1-v).(K_p^2-1)/(2.k_p).(p_i + c.\cot\phi).(R/a).^{(Kp-1)}.(R/r)^{(Kp+1)}$
$+ [(1-v).(K_p^2+1)/(2k_p) - v].(p_i + c.\cot\phi).(r/a)^{(Kp-1)}$

CASE 2: $\Psi = 0$, No dilation, $K_p = 1$

deformations: $u_r = r/(2G).\chi$

where $\chi = (2v-1).(p_z + c.\cot\phi) + (1-v).(K_p - 1).(p_i + c.\cot\phi).(R/a).^{(Kp-1)}.(R/r)^2 + (1-2.v).(p_i + c.\cot\phi).(r/a).^{(Kp-1)}$

Particular Solutions to Elastoplastic Problem - c-ϕ Material

CASE 3: $\phi = \phi$, and $c = 0$, Frictional Material

PLASTIC ZONE:

stresses: $\sigma_r = p_i.(r/a)^{Kp-1}$

$\sigma_\theta = p_i.K_p.(r/a)^{Kp-1}$

$\sigma_y = (\sigma_r + \sigma_\theta)/2 = p_i.[(1+Kp)/2].(r/a)^{Kp-1}$

deformations: $u_r = r/(2G).\chi$

for $\psi = \phi$
$\chi = (2v-1).p_z + (1-v).(K_p^2-1)/2.K_p.p_i.(R/a).^{(Kp-1)}.(R/r)^{(Kp+1)} + [(1-v).(K_p^2+1)/(2.K_p) - v].p_i.(r/a)^{(Kp-1)}$

for $\psi = 0$
$\chi = (2v-1).p_z + (1-v).(K_p - 1).p_i.(R/a).^{(Kp-1)}.(R/r)^2 + (1-2.v).pi.(r/a).^{(Kp-1)}$

8-4. Continuum Analyses Using Finite Difference, Finite Element, or Boundary Element Methods

Advances in continuum analysis techniques and the advent of fast, low-cost computers have led to the proliferation of continuum analysis programs aimed at the solution of a wide range of geomechanical problems including tunnel and shaft excavation and construction. For the purpose of this manual, continuum analyses refer to those methods or techniques that assume the rock medium to be a continuum and require the solution of a large set of simultaneous equations to calculate the states of stress and strain throughout the rock medium. The available techniques include the Finite Difference Method (FDM) (Cundall 1976), the Finite Element Method (FEM) (Bathe 1982), and the Boundary Element Method (BEM) (Venturini 1983). While there are subtle advantages of one method over another for some specialized applications, the three methods are equally useful for solving problems encountered in practice. Each of the three numerical techniques is used to solve an excavation problem in a rock medium whereby the field of interest is discretized and represented by a variety of elements. The changes in stress state and deformations are calculated at the element level given the (un)loading (construction) history and material properties. These numerical techniques provide the designer with powerful tools that can give unique insights into the tunnel/shaft support interaction problem during and after construction. Box 8-5 summarizes the steps followed in performing a continuum analysis. The following paragraphs describe these steps and how to consider continuum analyses as part of the design process. Advantages as well as the limitations of the numerical techniques are described.

Box 8-4. Elasto Plastic Particular Solution

Particular Solutions to Elastoplastic Problem

CASE 4: $\phi = 0$, $c = c$

yield condition: $p_z \geq p_i + c$

radius of yield zone: $R = a \,.\, \exp\,[\,(p_z\text{-}p_i)/(2.c) - 1/2\,]$

PLASTIC ZONE:

stresses: $\sigma_r = p_i + 2.c.\,\ln(r/a)$

$\sigma_\theta = p_i + 2.c.(1 + \ln(r/a))$

$\sigma_y = (\sigma_r + \sigma_\theta)/2 = p_i + c.(1 + 2.\ln(r/a))$

ELASTIC ZONE:

stresses: $\sigma_r = p_z - c.(a/r)^2 \,.\, \exp\,[(p_z - p_i)/c\ \ -1]$

$\sigma_\theta = p_z - c.(a/r)^2 \,.\, \exp\,[(p_z - p_i)/c\ \ -1]$

$\sigma_y = 2.v.p_z$

deformations:

$u_a = \quad c\,(1+v).[1 - c\,(1+v)/2.E\,].\ \exp\,[(p_z\text{-}p_i)/c\ \ -1] \cong [c(1+v)/E]\,.\ \exp\,[(p_z\text{-}p_i)/c\ -1]$

Box 8-5. Steps to Follow in Continuum Analysis of Tunnel and Shaft Excavations

1. Identify the need for and purpose of continuum analysis.
2. Define computer code requirements.
3. Modeling of the rock medium.
4. Two- and three-dimensional analyses.
5. Modeling of ground support and construction sequence.
6. Analysis approach.
7. Interpretation of analysis results.
8. Modification of support design and construction sequence, reanalysis.

a. Identify the need for and purpose of continuum analysis. The first step in carrying out a continuum analysis is identifying whether an analysis is needed. The FEM, FDM, or BEM numerical techniques are not substitutes for conventional methods of support design. The support system of a tunnel or shaft opening should first be selected using methods described in Chapters 7 and 9. The continuum analysis is then used to study the influence of the construction sequence and ground deformation on load transfer into supports. Safety factors and load factors commonly used in conventional methods should not be used in numerical analyses. Continuum analyses can incorporate details that cannot be accounted for using conventional methods such as inhomogeneous rock strata and nonuniform initial in situ stress, and hence provide guidance for modifications required in the support system. The continuum methods can best serve to improve support design through the opportunity they provide to study types

of situations from which general practical procedures can be developed (e.g., Hocking 1978). Modes of behavior that can be assessed using continuum analysis include the following:

(1) Elastic and elasto-plastic ground/support interaction. Convergence-confinement curves can be constructed using continuum analysis.

(2) Study of modes of failure.

(3) Identification of stress concentrations.

(4) Assessment of plastic zones requiring support.

(5) Analysis of monitoring data.

b. Define computer code requirements. A wide range of commercial and in-house programs are available for modeling tunnel and shaft construction. Prior to performing an analysis using a particular computer code, the user should determine the suitability of the program. Example analyses of problems for which a closed form solution is available (such as those given in Section 8-3) should be performed and the analysis results checked against those solutions. The user should verify that the program is capable of modeling the excavation process correctly and is able to represent the various support elements such as concrete and shotcrete lining, lattice girders, and bolts.

c. Modeling of the rock medium.

(1) The FEM, FDM, and BEM techniques model the rock mass as a continuum. This approximation is adequate when the rock mass is relatively free of discontinuities. However, these methods can still be used to model jointed rock masses by using equivalent material properties that reflect the strength reduction due to jointing (e.g., Zhu and Wang 1993; Pariseau 1993) or a material model that incorporates planes of weakness such as the Ubiquitous Joint Model (ITASCA 1992). Interface elements may be used to model displacements along discontinuities if they are deemed to be an important factor in the behavior of the system. The designer should first use as simple a model as possible and avoid adding details that may have little effect on the behavior of the overall system.

(2) The initial state of stress in the rock mass is important in determining the deformation due to excavation and the subsequent load carried by the support system. In a cross-anisotropic rock mass (in a horizontal topography) where material properties are constant in a horizontal plane, the state of stress can be described by a vertical stress component σ_v due to the weight of rock and a horizontal stress component $\sigma_h = K_0\sigma_v$. K_0 is the lateral in situ stress ratio. In situations where the rock mass is anisotropic, has nonhorizontal strata, or where the ground surface is inclined (e.g., sloping ground), methods such as those proposed by Amadei and Pan (1992) and Pan and Amadei (1993) should be used to establish the initial state of stress in the rock. Such methods are necessary because the initial stresses in the rock mass include nonzero shear stress components.

(3) The choice of a material model to represent the rock medium depends on the available properties obtained from laboratory and in situ testing programs and the required accuracy in the analysis. Many of the available continuum analysis programs have a large material model library that can be used. These include linear elastic and nonlinear elasto-plastic models and may have provisions to incorporate creep and thermal behavior. Available material/constitutive laws for modeling of the rock medium include the following:

- Linear Elastic.
- Non-Linear Elastic (Hyperbolic Model).
- Visco-Elastic.
- Elastic-plastic (Mohr-Coulomb failure criteria with an associated or nonassociated flow rule that controls material dilatancy, Hoek and Brown failure criteria).
- Elastic-viscoplastic.
- Bounding Surface Plasticity (Whittle 1987).

(4) The continuum analysis can be performed assuming either an effective stress or a total stress material behavior. Using effective stress behavior may be more appropriate for use in saturated rock masses and those of sedimentary origin such as shales or sandstones. There is sufficient evidence in the literature that would support the use of the effective stress law for some rocks (e.g., Warpinski and Teufel 1993; Berge, Wang, and Bonner 1993; Bellwald 1992). Examples of effective stress analysis of tunnels can be found in Cheng, Abousleiman, and Roegiers (1993).

(5) The size of the rock field (mesh size) and boundary conditions applied along the far-field edges of the model depend on the size of the opening and the hydrologic conditions. As a rule of thumb, the far-field

boundary is placed at a distance 5-10 times the size of the opening away from the centerline. Pore-pressure boundary conditions along the edges of the model and along the ground surface influence the predicted drawdown condition, pore-pressure buildup, and water inflow into the opening.

d. Two- and three-dimensional analyses. The available numerical techniques can be used to solve a shaft or tunnel excavation problem in two or three dimensions. Two-dimensional (2-D) analysis is appropriate for modeling tunnel sections along a running tunnel. Three-dimensional (3-D) analysis can be useful for understanding the behavior at tunnel and shaft intersections. However, 3-D analyses are laborious and involve the processing of large amounts of data. It is recommended that the analyst use a simplified 2-D model and arrive at a good understanding of the system response before commencing a full blown 3-D analysis. Examples of 2-D and 3-D analyses are given in Box 8-6 and Box 8-7.

e. Modeling of supports and construction sequence. The construction sequence of a tunnel/shaft is complicated and involves many details. It is not practical to incorporate all these details in the numerical simulation. Material removal and liner and dowel installation should be simplified into discrete steps. The following are a few examples of the possible simplifications:

(1) *Tunnel support.* Tunnel support can be cast-in-place concrete, precast concrete segments, shotcrete, or steel sets. The support can be modeled using the same types of elements used to model the rock, but using material models and properties that correspond to the support material. Since the thickness of the support is usually much less than the size of the opening, structural (beam) elements can be used to model the liner. In many situations, these elements are preferred as they better capture the bending behavior of the supports.

(2) *Shotcrete application.* There is usually a lag time between the application of shotcrete and the development of the full strength of the shotcrete. A simple approach to incorporate this effect into the continuum model would be to simulate shotcrete "installation" at the stage when the shotcrete develops its full strength.

(3) *Simulation of transfer of load to tunnel liner in a 2-D analysis.* During tunnel driving, support is installed close to the tunnel face. As the face is advanced, the rock relaxes further and load is applied to the supports. This problem is three-dimensional in nature. In a 2-D model, the rock is allowed to deform a percentage of its otherwise free deformation prior to "installation" of the support. This percentage ranges between 50 and 90 percent (Schwartz, Azzouz, and Einstein 1980) depending on how far the supports are installed behind the tunnel face. Section 8-2 discusses the development of deformations at the tunnel face in the context of the convergence-confinement method.

(4) *Fully grouted dowel with bearing plate.* The principal function of this support element is to reinforce the rock; the bearing plate has a relatively minor role in providing support for the overall system. In the numerical model, the bearing plate can be ignored; only a fully grouted dowel element needs to be represented.

(5) *Simulation of bolts and lattice girders in 2-D analysis.* Bolts and lattice girders are usually installed in a pattern in a tunnel/shaft section and at a specified spacing along the length of the excavation. Therefore, bolts and lattice girders are three-dimensional physical support components. In a 2-D analysis, the properties of bolts and lattice girders are "smeared" along the length of the tunnel. The properties of the bolts and lattice girders used in the model are equal to those of the actual supports averaged by the support spacing along the tunnel/shaft length (i.e., equivalent properties per unit length of tunnel/shaft).

f. Analysis approach. Throughout the process of constructing the model and performing the analyses, it is important to keep the number of details and analyses to a minimum. A well-defined set of parametric studies should be prepared and adjusted as the results of the analyses are examined. The analyst should maintain open communications with the design team. A common mistake is to expect the analysis to provide a resolution or accuracy higher than that of the input data.

g. Interpreting analysis results.

(1) Upon performing the first analysis, the analyst should carefully examine the results. The first step is to check whether the results are reasonable. Some of the questions that should be answered are as follows:

- Is the rock deforming as expected?

- Is the load distribution in the support system consistent with rock deformations?

- Is the change in the state of stress in the rock consistent with the failure criteria and other material properties?

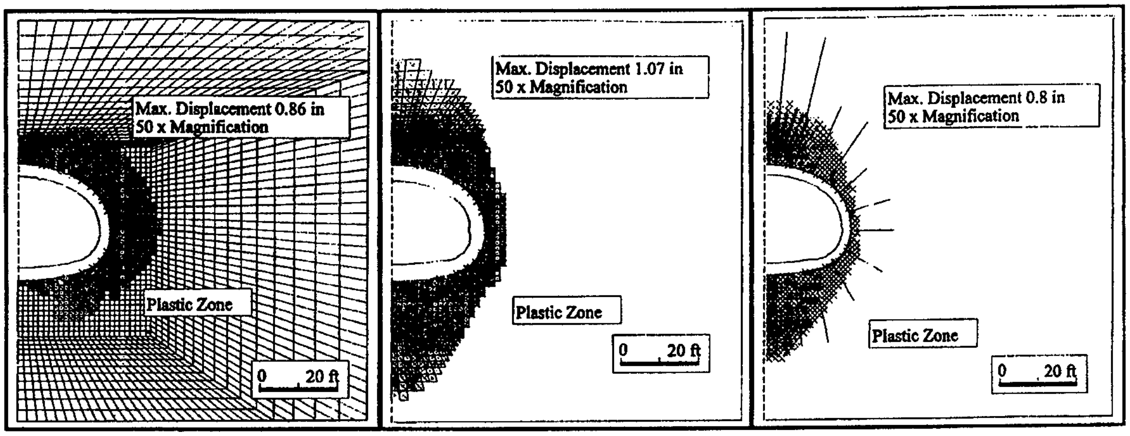
Box 8-6. Two-Dimensional Analysis of Elliptical Tunnel Section

Objective: Study the influence of initial in situ lateral stress ratio, K_0, on deformations and development of plastic zones around an elliptical tunnel section.

Rock Medium: Saturated Taylor Marl Shale, effective cohesion c' = 344 kPa and friction angle Phi' = 30°. Effective stress behavior, elastic-perfectly plastic material with a Mohr-Coulomb failure criteria

Support Type: Unsupported and supported with fully grouted dowels and 10-cm shotcrete lining.

Analysis Type: Finite Difference Analysis (FLAC Program, 2-D)



Deformation and yielded zones, $K_o = 1$ | **Deformation and yielded zones, $K_o = 1.5$** | **Deformation and yielded zones, $K_o = 1.5$**

Analysis Results: The increase in K_0 leads to an increase in the extent of the yielded zones in the crown and invert. Installation of dowels (longer dowels in the crown and invert compared with the springline) and the liner reduces the yielded zone.

Reference: Hashash, Y.M.A., and Cook, R. F. (1994) "Effective Stress Analysis of Supercollider Tunnels," 8th Int. Conf. Assoc. Comp. Methods and Advances in Rock Mechanics, Morgantown, West Virginia.

- Did the solution converge numerically?

Answering these and similar questions might reveal an error in the input data. A detailed check of the numerical results is necessary for the first analysis. A less rigorous check is required for subsequent analyses, but nonetheless the analyst should check for any possible anomalies in the results.

(2) Evaluation of the results of the continuum analyses and their implication regarding the rock-support interaction includes examining the following:

(a) *Deformations around the opening.* Deformations in the rock mass are related to the load transferred to the support system. Data from numerical analyses can be used to develop ground reaction curves (Section 8-2). Parametric studies can be used to develop general design charts that apply to more than one opening size or support configuration.

(b) *Loads in support system.* The analyses can provide moment, thrust, and shear force distributions in the liner. The data provided can be used to address possible modification in the liner, such as the introduction of pin connections to reduce excessive moments. Dowel load data can also be used to revise the distribution and modify the capacity of the proposed dowels. The analyses provide information on the influence of the opening on adjacent structures such as adjacent tunnels or surface buildings that may be distressed due to tunnel/shaft construction. Excessive deformations indicate the need for a more effective support system or a change in the construction method or sequence to mitigate potential damage.

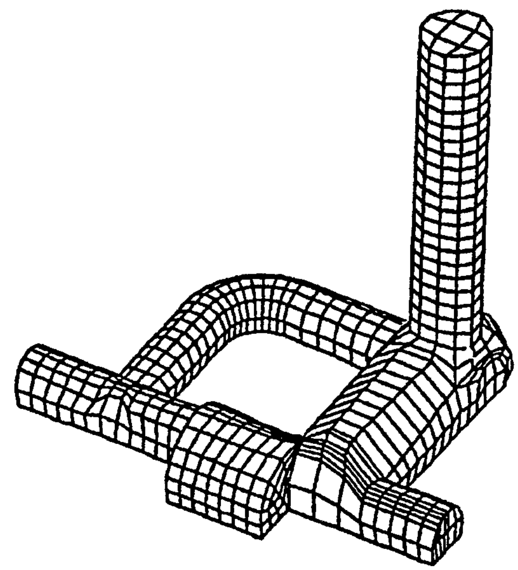
Box 8-7. Three-Dimensional Analysis of a Shaft and Tunnel Intersection

Objective: Study the stress distribution at shaft intersection with tunnel and ancillary galleries

Rock Medium: Eagle Ford Shale overlain by Austin Chalk. Total Stress behavior, linear elastic material

Support Type: No Support

Analysis Type: Finite Element Analysis (ABAQUS Program 3-D)



Analysis Results: Stress concentrations occur at tunnel/shaft intersections at zones experiencing a sudden change in geometry. The extent of the stress concentration is used to estimate the required dowel length in these areas.

Reference: Clark, G.T., and Schmidt, B. (1994) "Analysis and Design of SSC Underground Structures," Proceedings Boston Society of Civil Engineers.

(c) *Yielded and overstressed rock zones.* These zones indicate a potential for rock spalling and rock falls if located near the excavated surface. Large yielded zones indicate a general weakening of the rock and the need to provide reinforcement. The zones can be used to size rock reinforcements (bolts and dowels).

(d) *Pore-pressure distribution and water inflow.* This will provide information on the direction of potential water flow, as well as the expected changes in pore pressures in the rock. The information is relevant in rock masses with discontinuities, as well as in swelling rocks. Contours of pore-pressure distribution are useful in this regard. Many of the commercially available codes have postprocessors that provide the user with a wide range of output capabilities including tabulated data, contour plots, deformed mesh plots, and color graphics. These are useful tools that can convey the results of the analysis in a concise manner especially to outside reviewers.

h. Modification of support system, reanalysis. Continuum analyses provide insight into the behavior of the overall support system and the adequacy of the support system. The analyses may highlight some deficiencies or possible overdesign in the proposed support system. Several analysis iterations may be required to optimize the design.

i. Limitations of continuum analyses. Continuum analysis techniques are versatile tools that provide much understanding of problems involving underground structures. However, they have several limitations that have to be considered to use these techniques effectively. Continuum analysis techniques are not a substitute for conventional design techniques and sound engineering judgement. A continuum analysis cannot give warning of phenomena such as localized spalling. Continuum analysis in geotechnical applications is vastly different from applications in the structural field. Continuum analysis in structural application is geared to satisfy code requirements where the parameters are well defined. Continuum analysis in geotechnical and underground applications involves many unknown factors and requires much judgement on the part of the user. The complexity of a continuum analysis is often limited by the availability of geomechanics data and rock properties. The designer should avoid making too many assumptions regarding the material properties in a model while still expecting to obtain useful information from the analysis. Continuum analyses predict stresses, strains, and displacements but generally do not tell anything about stability and safety factors. Some specialized programs can provide predictions of stability (e.g, Sloan 1981).

j. Example applications. Boxes 8-7 and 8-8 illustrate the use of continuum analyses for shaft and tunnel problems as applied to the Superconducting Super Collider underground structures.

8-5. Discontinuum Analyses

Closed form solutions and continuum analyses of tunnel and shaft problems in rock ignore weaknesses and flaws that interrupt the continuity of the rock mass. The presence of weaknesses makes the rock a collection of tightly fitted blocks. The rock, thus, exhibits a behavior different from a continuous material. This section describes approaches to analysis of openings in rock behaving as a discontinuum.

a. Key block theory.

(1) The best known theory for discontinuous analysis of rocks is the key block theory pioneered by Goodman and Shi (1985). In a key block analysis, the object is to find the critical blocks created by intersections of discontinuities in a rock mass excavated along defined surfaces. The analysis can skip over many combinations of joints and proceed directly to consider certain critical (key) blocks. If these blocks are stabilized, no other blocks can fall into the opening. The principal assumptions are as follows:

(a) All joint surfaces are planar. Linear vector analysis can therefore be used for the solution of the problem.

(b) Joint surfaces extend through the entire volume of the rock mass. No discontinuities terminate within a block. No new discontinuities can develop due to cracking.

(c) The intact blocks defined by the discontinuities are rigid. Deformations are due to block movement but not block deformation.

(d) The discontinuity and excavation surfaces are defined. If the joint set orientations are actually dispersed about a central tendency, one direction must be chosen to represent the set.

(2) Figure 8-5 illustrates the concept of key block analysis. Block analysis can be carried out using stereographic projection graphical methods or vector methods. Hatzor and Goodman (1993) illustrate the application of the analysis to the Hanging Lake Tunnel, Glenwood Canyon, Colorado. The analysis methods have been incorporated into computer programs.

b. Discrete element methods.

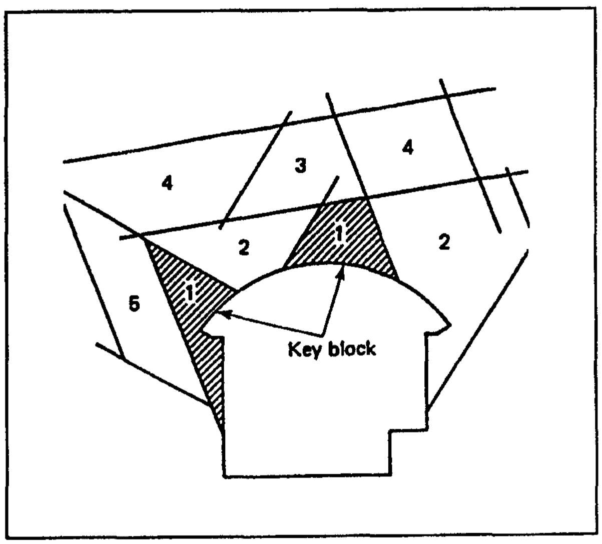


Figure 8-5. Key block analysis

(1) Cundall and Hart (1993) propose that the term discrete element method applies to computer methods that allow finite displacements and rotations of discrete bodies, including complete detachment, and recognize new contacts automatically as the calculation progresses. Four main classes of computer methods conform to this definition:

(a) *Distinct element methods.* They use explicit, time-marching to solve the equations of motion directly. Bodies may be rigid or deformable; contacts are deformable.

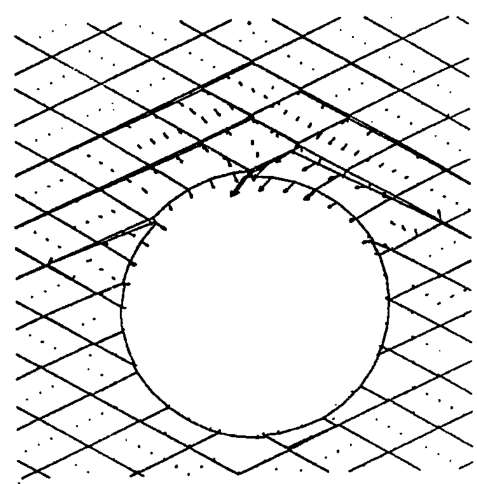
(b) *Modal methods.* They are similar to distinct element methods in the case of rigid bodies, but for deformable bodies, modal superposition is used.

(c) *Discontinuous deformation methods.* In these methods, contacts are rigid, and bodies may be rigid or deformable.

(d) *Momentum-exchange methods.* In these methods, both the contacts and the bodies are rigid; momentum is exchanged between two contacting bodies during an instantaneous collision. Frictional sliding can be represented.

(2) Figure 8-6 shows an analysis of a tunnel opening in a jointed rock mass using the distinct element method and the computer program UDEC.

(3) The block theory and discrete element analysis methods are useful in identifying unstable blocks in large underground chambers. In smaller openings such as shafts and tunnels, they are less useful. Cost considerations may preclude the use of discontinuum analysis in small openings due to budget constraints. Large openings that are used to house expensive equipment have big enough budgets to perform these analyses. Discontinuum analysis methods are limited by the unavailability of sufficient data during design. The methods can be used during construction after mapping of discontinuities to identify potential unstable blocks that require support (NATM).



Movement of blocks around tunnel; velocities denoted by arrows

Figure 8-6. Distinct element analysis, Cundell and Hart 1993

Chapter 9
Design of Permanent, Final Linings

Most tunnels and shafts in rock are furnished with a final lining. The common options for final lining include the following:

- Unreinforced concrete.
- Reinforced concrete.
- Segments of concrete.
- Steel backfilled with concrete or grout.
- Concrete pipe with backfill.

In many respects, tunnel and shaft lining design follows rules different from standard structural design rules. An understanding of the interaction between rock and lining material is necessary for tunnel and shaft lining design.

9-1. Selection of a Permanent Lining

The first step in lining design is to select the appropriate lining type based on the following criteria:

- Functional requirements.
- Geology and hydrology.
- Constructibility.
- Economy.

It may be necessary to select different lining systems for different lengths of the same tunnel. For example, a steel lining may be required for reaches of a pressure tunnel with low overburden or poor rock, while other reaches may require a concrete lining or no lining at all. A watertight lining may be required through permeable shatter zones or through strata with gypsum or anhydrite, but may not be required for the remainder of the tunnel. Sometimes, however, issues of constructibility will make it appropriate to select the same lining throughout. For example, a TBM tunnel going through rock of variable quality, may require a concrete segmental lining or other substantial lining in the poor areas. The remainder of the tunnel would be excavated to the same dimension, and the segmental lining might be carried through the length of the tunnel, especially if the lining is used as a reaction for TBM propulsion jacks.

a. Unlined tunnels. In the unlined tunnel, the water has direct access to the rock, and leakage will occur into or out of the tunnel. Changes in pressure can cause water to pulse in and out of a fissure, which in the long term can wash out fines and result in instability. This can also happen if the tunnel is sometimes full, sometimes empty, as for example a typical flood control tunnel. Metal ground support components can corrode, and certain rock types suffer deterioration in water, given enough time. The rough surface of an unlined tunnel results in a higher Mannings number, and a larger cross section may be required than for a lined tunnel to meet hydraulic requirements. For an unlined tunnel to be feasible, the rock must be inert to water, free of significant filled joints or faults, able to withstand the pressures in the tunnel without hydraulic jacking or other deleterious effects, and be sufficiently tight that leakage rates are acceptable. Norwegian experience indicates that typical unlined tunnels leak between 0.5 and 5 l/s/km (2.5-25 gpm/1,000 ft). Bad rock sections in an otherwise acceptable formation can be supported and sealed locally. Occasional rock falls can be expected, and rock traps to prevent debris from entering valve chambers or turbines may be required at the hydropower plant. Unlined tunnels are usually furnished with an invert pavement, consisting of 100-300 mm (4-12 in.) of unreinforced or nominally reinforced concrete, to provide a suitable surface for maintenance traffic and to decrease erosion.

b. Shotcrete lining. A shotcrete lining will provide ground support and may improve leakage and hydraulic characteristics of the tunnel. It also protects the rock against erosion and deleterious action of the water. To protect water-sensitive ground, the shotcrete should be continuous and crack-free and reinforced with wire mesh or fibers. As with unlined tunnels, shotcrete-lined tunnels are usually furnished with a cast-in-place concrete invert.

c. Unreinforced concrete lining. An unreinforced concrete lining primarily is placed to protect the rock from exposure and to provide a smooth hydraulic surface. Most shafts that are not subject to internal pressure are lined with unreinforced concrete. This type of lining is acceptable if the rock is in equilibrium prior to the concrete placement, and loads on the lining are expected to be uniform and radial. An unreinforced lining is acceptable if leakage through minor shrinkage and temperature cracks is acceptable. If the groundwater is corrosive to concrete, a tighter lining may be required to prevent corrosion by the seepage water. An unreinforced lining is generally not acceptable through soil overburden or in badly squeezing rock, which can exert nonuniform displacement loads.

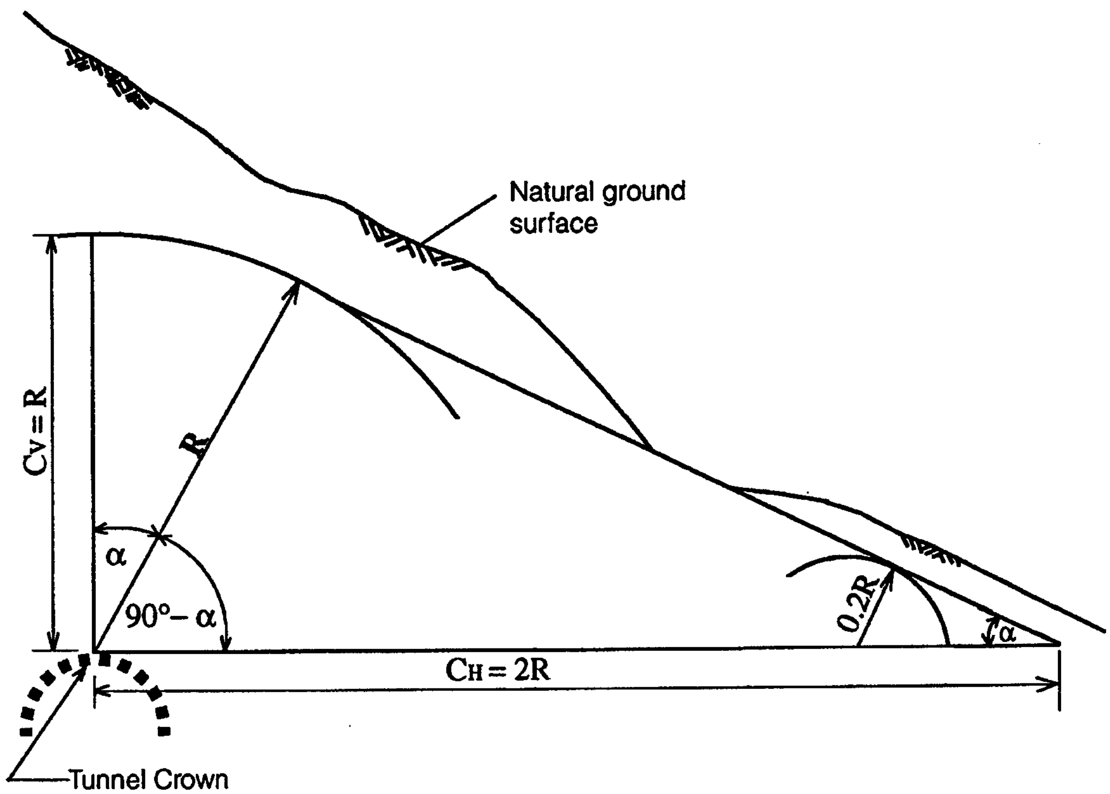
d. Reinforced concrete linings. The reinforcement layer in linings with a single layer should be placed close to the inside face of the lining to resist temperature stresses and shrinkage. This lining will remain basically undamaged for distortions up to 0.5 percent, measured as diameter change/diameter, and can remain functional for greater distortions. Multiple layers of reinforcement may be required due to large internal pressures or in a squeezing or swelling ground to resist potential nonuniform ground displacements with a minimum of distortion. It is also used where other circumstances would produce nonuniform loads, in rocks with cavities. For example, nonuniform loads also occur due to construction loads and other loads on the ground surface adjacent to shafts; hence, the upper part of a shaft lining would often require two reinforcement layers. Segmental concrete linings are often required for a tunnel excavated by a TBM. See Section 5-3 for details and selection criteria.

e. Pipe in tunnel. This method may be used for conduits of small diameter. The tunnel is driven and provided with initial ground support, and a steel or concrete pipe with smaller diameter is installed. The void around the pipe is then backfilled with lean concrete fill or, more economically, with cellular concrete. The pipe is usually concrete pipe, but steel may be required for pressure pipe. Plastic, fiber-reinforced plastic, or ceramic or clay pipes have also been used.

f. Steel lining. Where the internal tunnel pressure exceeds the external ground and groundwater pressure, a steel lining is usually required to prevent hydro-jacking of the rock. The important issue in the design of pressurized tunnels is confinement. Adequate confinement refers to the ability of a rock mass to withstand the internal pressure in an unlined tunnel. If the confinement is inadequate, hydraulic jacking may occur when hydraulic pressure within a fracture, such as a joint or bedding plane, exceeds the total normal stress acting across the fracture. As a result, the aperture of the fracture may increase significantly, yielding an increased hydraulic conductivity, and therefore increased leakage rates. General guidance concerning adequate confinement is that the weight of the rock mass measured vertically from the pressurized waterway to the surface must be greater than the internal water pressure. While this criterion is reasonable for tunneling below relatively level ground, it is not conservative for tunnels in valley walls where internal pressures can cause failure of sidewalls. Sidewall failure occurred during the development of the Snowy Mountains Projects in Australia. As can be seen from Figure 9-1, the Snowy Mountains Power Authority considered that side cover is less effective in terms of confinement as compared with vertical cover. Figure 9-2 shows guidance developed in Norway after several incidents of sidewall failure had taken place that takes into account the steepness of the adjacent valley wall. According to Electric Power Research Institute (EPRI) (1987), the Australian and the Norwegian criteria, as outlined in Figures 9-1 and 9-2, usually are compatible with actual project performance. However, they must be used with care, and irregular topographic noses and surficial deposits should not be considered in the calculation of confinement. Hydraulic jacking tests or other stress measurements should be performed to confirm the adequacy of confinement.

g. Lining leakage. It must be recognized that leakage through permeable geologic features can occur despite adequate confinement, and that leakage through discontinuities with erodible gouge can increase with time. Leakage around or through concrete linings in gypsum, porous limestone, and in discontinuity fillings containing porous or flaky calcite can lead to cavern formation and collapse. Leakage from pressured waterways can lead to surface spring formation, mudslides, and induced landslides. This can occur when the phreatic surface is increased above the original water table by filling of the tunnel, the rock mass is permeable, and/or the valleyside is covered by less permeable materials.

h. Temporary or permanent drainage. It may not be necessary or reasonable to design a lining for external water pressure. During operations, internal pressures in the tunnel are often not very different from the in situ formation water pressure, and leakage quantities are acceptable. However, during construction, inspection, and maintenance, the tunnel must be drained. External water pressure can be reduced or nearly eliminated by providing drainage through the lining. This can be accomplished by installing drain pipes into the rock or by applying filter strips around the lining exterior, leading to drain pipes. Filter strips and drains into the ground usually cannot be maintained; drain collectors in the tunnel should be designed so they can be flushed and cleaned. If groundwater inflows during construction are too large to handle, a grouting program can be instituted to reduce the flow. The lining should be designed to withstand a proportion of the total external water pressure because the drains cannot reduce the pressures to zero, and there is always a chance that some drains will clog. With proper drainage, the design water pressure may be taken as the lesser of 25 percent of the full pressure and a pressure equivalent to a column of water three tunnel diameters high. For construction conditions, a lower design pressure can be chosen.



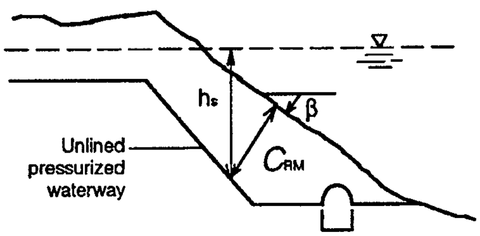
Current practice, equivalent cover. From *Unlined Tunnels of the Snowy Mountains,* Hydroelectric Authority, ASCE Conference, Oct. 1963.

Figure 9-1. Snowy Mountains criterion for confinement

9-2. General Principles of Rock-Lining Interaction

The most important material for the stability of a tunnel is the rock mass, which accepts most or all of the distress caused by the excavation of the tunnel opening by redistributing stress around the opening. The rock support and lining contribute mostly by providing a measure of confinement. A lining placed in an excavated opening that has reached stability (with or without initial rock support) will experience no stresses except due to self-weight. On the other hand, a lining placed in an excavated opening in an elastic rock mass at the time that 70 percent of all latent motion has taken place will experience stresses from the release of the remaining 30 percent of displacement. The actual stresses and displacements will depend on the modulus of the rock mass and that of the tunnel lining material. If the modulus or the in situ stress is anisotropic, the lining will distort, as the lining material deforms as the rock relaxes. As the lining material pushes against the rock, the rock load increases.

a. Failure modes for concrete linings. Conventional safety factors are the ratio between a load that causes failure or collapse of a structure and the actual or design load (capacity/load or strength/stress). The rock load on tunnel ground support depends on the interaction between the rock and the rock support, and overstress can often be alleviated by making the rock support more flexible. It is possible to redefine the safety factor for a lining by the ratio of the



C_{RM} = minimum rock cover = $h_s \gamma_W F / \gamma_R \cos\beta$;
h_s = static head; γ_W = unit weight of water;
γ_R = unit weight of rock; β = slope angle
(varies along slope); F = safety factor

Figure 9-2. Norwegian criterion for confinement

stress that would cause failure and the actual induced stress for a particular failure mechanism. Failure modes for concrete linings include collapse, excessive leakage, and accelerated corrosion. Compressive yield in reinforcing steel or concrete is also a failure mode; however, tension cracks in concrete usually do not result in unacceptable performance.

b. Cracking in tunnel or shaft lining. A circular concrete lining with a uniform external load will experience a uniform compressive stress (hoop stress). If the lining is subjected to a nonuniform load or distortion, moments will develop resulting in tensile stresses at the exterior face of the lining, compressive stresses at the interior face at some points, and tension at other points. Tension will occur if the moment is large enough to overcome the hoop compressive stress in the lining and the tensile strength of the concrete is exceeded. If the lining were free to move under the nonuniform loading, tension cracks could cause a collapse mechanism. Such a collapse mechanism, however, is not applicable to a concrete lining in rock; rock loads are typically not following loads, i.e., their intensity decreases as the lining is displaced in response to the loads; and distortion of the lining increases the loads on the lining and deformation toward the surrounding medium. These effects reduce the rock loads in highly stressed rock masses and increase them when stresses are low, thus counteracting the postulated failure mechanism when the lining has flexibility. Tension cracks may add flexibility and encourage a more uniform loading of the lining. If tension cracks do occur in a concrete lining, they are not likely to penetrate the full thickness of the lining because the lining is subjected to radial loads and the net loads are compressive. If a tension crack is created at the inside lining face, the cross-section area is reduced resulting in higher compressive stresses at the exterior, arresting the crack. Tension cracks are unlikely to create loose blocks. Calculated tension cracks at the lining exterior may be fictitious because the rock outside the concrete lining is typically in compression, and shear bond between concrete and rock will tend to prevent a tension crack in the concrete. In any event, such tension cracks have no consequence for the stability of the lining because they cannot form a failure mechanism until the lining also fails in compression. The above concepts apply to circular linings. Noncircular openings (horseshoe-shaped, for example) are less forgiving, and tension cracks must be examined for their contribution to a potential failure mode, especially when generated by following loads.

c. Following loads. Following loads are loads that persist independently of displacement. The typical example is the hydrostatic load from formation water. Fortunately the hydrostatic load is uniform and the circular shape is ideal to resist this load. Other following loads include those resulting from swelling and squeezing rock displacements, which are not usually uniform and can result in substantial distortions and bending failure of tunnel linings.

9-3. Design Cases and Load Factors for Design

The requirements of EM 1110-2-2104 shall apply to the design of concrete tunnels untless otherwise stated herein. Selected load factors for water tunnels are shown in Table 9-1. These load factors are, in some instances, different from load factors used for surface structures in order to consider the particular environment and behavior of underground structures. On occasion there may be loads other than those shown in Table 9-1, for which other design cases and load factors must be devised. Combinations of loads other than those shown may produce less favorable conditions. Design load cases and factors should be carefully evaluated for each tunnel design.

9-4. Design of Permanent Concrete Linings

Concrete linings required for tunnels, shafts, or other underground structures must be designed to meet functional criteria for water tightness, hydraulic smoothness, durability, strength, appearance, and internal loads. The lining must also be designed for interaction with the surrounding rock mass and the hydrologic regime in the rock and consider constructibility and economy.

Table 9-1
Design Cases and Recommended Load Factors for Water Tunnel[1]

Load	1	2	3	4
Dead load[2]	1.3	1.1	1.1	1.1
Rock load[3]	1.4	1.2	1.4	1.2
Hydrostatic operational[4]	1.4	-	-	-
Hydrostatic transient[5]	-	1.1	-	-
Hydrostatic external[6]	-	-	1.4	1.4
Live load				1.4

[1] This table applies to reinforced concrete linings.
[2] Self-weight of the lining, plus the weight of permanent fixtures, if any. Live load, for example, vehicles in the tunnel, would generally have a load factor of 1.4. In water tunnels, this load is usually absent during operations.
[3] Rock loads are the loads and/or distortions derived from rock-structure interaction assessments.
[4] Maximum internal pressure, minus the minimum external water pressure, under normal operating conditions.
[5] Maximum transient internal pressure, for example, due to water hammer, minus the minimum external water pressure.
[6] Maximum groundwater pressure acting on an empty tunnel.
Note: The effects of net internal hydrostatic loads on the concrete lining may be reduced or eliminated by considering interaction between lining and the surrounding rock, as discussed in Section 8-5.

a. Lining thickness and concrete cover over steel. For most tunnels and shafts, the thickness of concrete lining is determined by practical constructibility considerations rather than structural requirements. Only for deep tunnels required to accept large external hydrostatic loads, or tunnels subjected to high, nonuniform loads or distortions, will structural requirements govern the tunnel lining thickness. For concrete placed with a slick-line, the minimum practical lining thickness is about 230 mm (9 in.), but most linings, however, require a thickness of 300 mm (12 in.) or more. Concrete clear cover over steel in underground water conveyance structures is usually taken as 100 mm (4 in.) where exposed to the ground and 75 mm (3 in.) for the inside surface. These thicknesses are greater than normally used for concrete structures and allow for misalignment during concrete placement, abrasion and cavitation effects, and long-term exposure to water. Tunnels and other underground structures exposed to aggressive corrosion or abrasion conditions may require additional cover. EM 1110-2-2104 provides additional guidance concerning concrete cover.

b. Concrete mix design. EM 1110-2-2000 should be followed in the selection of concrete mix for underground works. Functional requirements for underground concrete and special constructibility requirements are outlined below. For most underground work, a 28-day compressive strength of 21 MPa (3,000 psi) and a water/cement ratio less than 0.45 is satisfactory. Higher strengths, up to about 35 MPa (5,000 psi) may be justified to achieve a thinner lining, better durability or abrasion resistance, or a higher modulus. One-pass segmental linings may require a concrete strength of 42 MPa (6,000 psi) or higher. Concrete for tunnel linings is placed during the day, cured overnight, and forms moved the next shift for the next pour. Hence, the concrete may be required to have attained sufficient strength after 12 hr to make form removal possible. The required 12-hr strength will vary depending on the actual loads on the lining at the time of form removal. Concrete must often be transported long distances through the tunnel to reach the location where it is pumped into the lining forms. The mix design must result in a pumpable concrete with a slump of 100 to 125 mm (4 to 5 in.) often up to 90 min after mixing. Accelerators may be added and mixed into the concrete just before placement in the lining forms. Functionality, durability, and workability requirements may conflict with each other in the selection of the concrete mix. Testing of trial mixes should include 12-hr strength testing to verify form removal times.

c. Reinforcing steel for crack control. The tensile strain in concrete due to curing shrinkage is of the order of 0.05 percent. Additional tensile strains can result from long-term exposure to the atmosphere (carbonization and other effects) and temperature variations. In a tunnel carrying water, these long-term effects are generally small. Unless cracking due to shrinkage is controlled, the cracks will occur at a few discrete locations, usually controlled by variations in concrete thickness, such as rock overbreak areas or at steel rib locations. The concrete lining is cast against a rough rock surface, incorporating initial ground support elements such as shotcrete, dowels, or steel sets; therefore, the concrete is interlocked with the rock in the longitudinal direction. Incorporation of expansion joints therefore has little effect on the formation and control of cracks. Concrete linings should be placed without expansion joints, and reinforcing steel should be continued across construction joints. Tunnel linings have been constructed using concrete with polypropylene olefin or steel fibers for crack control in lieu of reinforcing steel. Experience with the use of fibers for this purpose, however, is limited at the time of this writing. In tunnels, shrinkage reinforcement is usually 0.28 percent of the cross-sectional area. For

highly corrosive conditions, up to 0.4 percent is used. Where large overbreaks are foreseen in a tunnel excavated by blasting, the concrete thickness should be taken as the theoretical concrete thickness plus one-half the estimated typical overbreak dimension.

d. Concrete linings for external hydrostatic load. Concrete linings placed without provisions for drainage should be designed for the full formation water pressure acting on the outside face. If the internal operating pressure is greater than the formation water pressure, the external water pressure should be taken equal to the internal operating pressure, because leakage from the tunnel may have increased the formation water pressure in the immediate vicinity of the tunnel. If the lining thickness is less than one-tenth the tunnel radius, the concrete stress can be found from the equation

$$f_c = pR/t \qquad (9\text{-}1)$$

where

f_c = stress in concrete lining

p = external water pressure

R = radius to circumferential centerline of lining

t = lining thickness

For a slender lining, out-of-roundness should be considered using the estimated radial deviation from a circular shape u_o. The estimated value of u_o should be compatible with specified roundness construction tolerances for the completed lining.

$$f_c = pR/t \pm 6pRu_o/(t^2\ (1-p/p_{cr})) \qquad (9\text{-}2)$$

where p_{cr} is the critical buckling pressure determined by Equation 9-3.

$$p_{cr} = 3EI/R^3 \qquad (9\text{-}3)$$

If the lining thickness is greater than one-tenth the tunnel radius, a more accurate equation for the maximum compressive stress at the inner surface is

$$f_c = 2pR_2^2/(R_2^2 - R_1^2) \qquad (9\text{-}4)$$

where

R_2 = radius to outer surface

R_1 = radius to inner surface of lining

e. Circular tunnels with internal pressure. Analysis and design of circular, concrete-lined rock tunnels with internal water pressure require consideration of rock-structure interaction as well as leakage control.

(1) *Rock-structure interaction.* For thin linings, rock-structure interaction for radial loads can be analyzed using simplified thin-shell equations and compatibility of radial displacements between lining and rock. Consider a lining of average radius, a, and thickness, t, subject to internal pressure, p_i, and external pressure, p_r, where Young's modulus is E_c and Poisson's Ratio is ν_c. The tangential stress in the lining is determined by Equation 9-5.

$$\sigma_t = (p_i - p_r)a/t \qquad (9\text{-}5)$$

and the relative radial displacement, assuming plane strain conditions, is shown in Equation 9-6.

$$\Delta a/a = (p_i - p_r)\ (a/t)\ ((1-\nu_c^2)/E_c) = (p_i - p_r)\ K_c \qquad (9\text{-}6)$$

The relative displacement of the rock interface for the internal pressure, p_r, assuming a radius of a and rock properties E_r and ν_r, is determined by Equation 9-7.

$$\Delta a/a = p_r(1 + \nu_r)/E_r = p_rK_r \qquad (9\text{-}7)$$

Setting Equations 9-6 and 9-7 equal, the following expression for p_r is obtained:

$$p_r = p_i\ K_c/(K_c + K_r) \qquad (9\text{-}8)$$

From this is deduced the net load on the lining, $p_i - p_r$, the tangential stress in the lining, G_t, and the strain and/or relative radial displacement of the lining:

$$\varepsilon = \Delta\ a/a = (p_i/E_c)(a/t)(K_r/(K_r + K_c)) \qquad (9\text{-}9)$$

For thick linings, more accurate equations can be developed from thick-walled cylinder theory. However, considering the uncertainty of estimates of rock mass modulus,

the increased accuracy of calculations is usually not warranted.

(2) *Estimates of lining leakage*. The crack spacing in reinforced linings can be estimated from

$$s = 5(d - 7.1) + 33.8 + 0.08\ d\rho(mm) \qquad (9\text{-}10)$$

where d is the diameter of the reinforcing bars and ρ is the ratio of steel area to concrete area, A_s/A_c. For typical tunnel linings, s is approximately equal to 0.1 d/ρ. The average crack width is then $w = s\ \varepsilon$. The number of cracks in the concrete lining can then be estimated as shown in Equation 9-11.

$$n = 2\ \pi\ a/s \qquad (9\text{-}11)$$

The quantity of water flow through n cracks in a lining of thickness t per unit length of tunnel can be estimated from Equation 9-12.

$$q = (n/2\eta)(\Delta p/t)\ w^3 \qquad (9\text{-}12)$$

where η is the dynamic viscosity of water, and Δp is the differential water pressure across the lining. If the lining is crack-free, the leakage through the lining can be estimated from Equation 9-13.

$$q = 2\ \pi\ a\ k_c\ \Delta p/\gamma_w\ t \qquad (9\text{-}13)$$

where k_c is the permeability of the concrete.

(3) *Acceptability of lining leaking*. The acceptability of leakage through cracks in the concrete lining is dependent on an evaluation of at least the following factors.

- Acceptability of loss of usable water from the system.

- Effect on hydrologic regime. Seepage into underground openings such as an underground powerhouse, or creation of springs in valley walls or lowering of groundwater tables may not be acceptable.

- Rock formations subject to erosion, dissolution, swelling, or other deleterious effects may require seepage and crack control.

- Rock stress conditions that can result in hydraulic jacking may require most or all of the hydraulic pressure to be taken by reinforcement or by an internal steel lining.

It may be necessary to assess the effects of hydraulic interaction between the rock mass and the lining. If the rock is very permeable relative to the lining, most of the driving pressure difference is lost through the lining; leakage rates can be controlled by the lining. If the rock is tight relative to the lining, then the pressure loss through the lining is small, and leakage is controlled by the rock mass. These factors can be analyzed using continuity of water flow through lining and ground, based on the equations shown above and in Chapter 3. When effects on the groundwater regime (rise in groundwater table, formation of springs, etc.) are critical, conditions can be analyzed with the help of computerized models.

f. Linings subject to bending and distortion. In most cases, the rock is stabilized at the time the concrete lining is placed, and the lining will accept loads only from water pressure (internal, external, or both). However, reinforced concrete linings may be required to be designed for circumferential bending in order to minimize cracking and avoid excessive distortions. Box 9-1 shows some general recommendations for selection of loads for design. Conditions causing circumferential bending in linings are as follows:

- Uneven support caused a thick layer of rock of much lower modulus than the surrounding rock, or a void left behind the lining.

- Uneven loading caused by a volume of rock loosened after construction, or a localized water pressure trapped in a void behind the lining.

- Displacements from uneven swelling or squeezing rock.

- Construction loads, such as from nonuniform grout pressures.

Bending reinforcement may also be required through shear zones or other zones of poor rock, even though the remainder of the tunnel may have received no reinforcement or only shrinkage reinforcement. There are many different methods available to analyze tunnel linings for bending and distortion. The most important types can be classified as follows:

Box 9-1. General Recommendations for Loads and Distortions

1. Minimum loading for bending: Vertical load uniformly distributed over the tunnel width, equal to a height of rock 0.3 times the height of the tunnel.

2. Shatter zone previously stabilized: Vertical, uniform load equal to 0.6 times the tunnel height.

3. Squeezing rock: Use pressure of 1.0 to 2.0 times tunnel height, depending on how much displacement and pressure relief is permitted before placement of concrete. Alternatively, use estimate based on elastoplastic analysis, with plastic radius no wider than one tunnel diameter.

4. For cases 1, 2, and 3, use side pressures equal to one-half the vertical pressures, or as determined from analysis with selected horizontal modulus. For excavation by explosives, increase values by 30 percent.

5. Swelling rock, saturated in situ: Use same as 3 above.

6. Swelling rock, unsaturated or with anhydrite, with free access to water: Use swell pressures estimated from swell tests.

7. Noncircular tunnel (horseshoe): Increase vertical loads by 50 percent.

8. Nonuniform grouting load, or loads due to void behind lining: Use maximum permitted grout pressure over area equal to one-quarter the tunnel diameter, maximum 1.5 m (5 ft).

- Free-standing ring subject to vertical and horizontal loads (no ground interaction).

- Continuum mechanics, closed solutions.

- Loaded ring supported by springs simulating ground interaction (many structural engineering codes).

- Continuum mechanics, numerical solutions.

The designer must select the method which best approximates the character and complexity of the conditions and the tunnel shape and size.

(1) *Continuum mechanics, closed solutions.* Moments developed in a lining are dependent on the stiffness of the lining relative to that of the rock. The relationship between relative stiffness and moment can be studied using the closed solution for elastic interaction between rock and lining. The equations for this solution are shown in Box 9-2, which also shows the basic assumptions for the solution. These assumptions are hardly ever met in real life except when a lining is installed immediately behind the advancing face of a tunnel or shaft, before elastic stresses have reached a state of plane strain equilibrium. Nonetheless, the solution is useful for examining the effects of variations in important parameters. It is noted that the maximum moment is controlled by the flexibility ratio

$$\alpha = E_r R^3/(E_c)I \qquad (9\text{-}14)$$

For a large value of α (large rock mass modulus), the moment becomes very small. Conversely, for a small value (relatively rigid lining), the moment is large. If the rock mass modulus is set equal to zero, the rock does not restrain the movement of the lining, and the maximum moment is

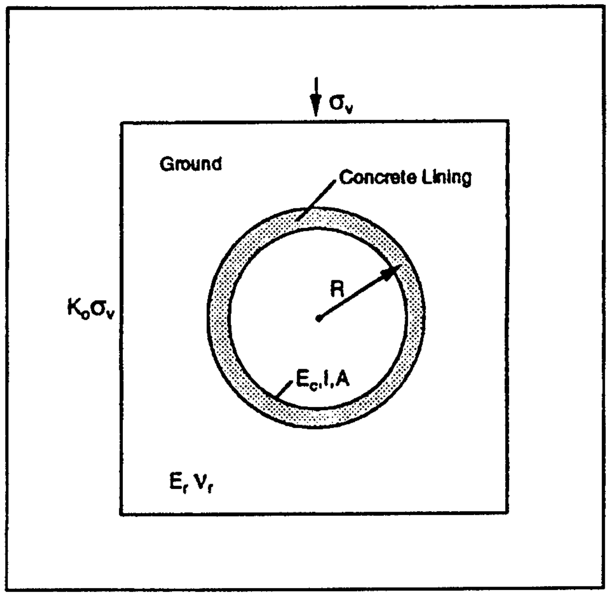
$$M = 0.25\sigma_v(1 - K_o)R^2 \qquad (9\text{-}15)$$

With $K_o = 1$ (horizontal and vertical loads equal), the moment is zero; with $K_o = 0$ (corresponding to pure vertical loading of an unsupported ring), the largest moment is obtained. A few examples will show the effect of the flexibility ratio. Assume a concrete modulus of 3,600,000 psi, lining thickness 12 in. ($I = 12^3/12$), rock mass modulus 500,000 psi (modulus of a reasonably competent limestone), $\nu_r = 0.25$, and tunnel radius of 72 in.; then α = 360, and the maximum moment

$$M = 0.0081 \times \sigma_v(1 - K_o)R^2 \qquad (9\text{-}16)$$

This is a very small moment. Now consider a relatively rigid lining in a soft material: Radius 36 in., thickness 9 in., and rock mass modulus 50,000 psi (a soft shale or crushed rock); then the maximum moment is

Box 9-2. Lining in Elastic Ground, Continuum Model



Assumptions:

Plane strain, elastic radial lining pressures are equal to in situ stresses, or a proportion thereof

Includes tangetial bond between lining and ground

Lining distortion and ocmpression resisted/relieved by ground reactions

Maximum/minimum bending movement

$$M = \pm\sigma_v\,(1 - K_o)\; R^2/(4 + \frac{3 - 2\nu_r}{3\,(1 + \nu_r)\,(1 + \nu_r)} \cdot \frac{E_r\,R^3}{E_c\,I}$$

Maximum/minimum hoop force

$$N = \sigma_v\,(1 + K_o)\; R/(2 + (1 - K_o)\,\frac{2(1 - \nu_r)}{(1 - 2\nu_r)\,(1 + \nu)} \cdot \frac{E_r R}{E_c A}) \pm \sigma_v\,(1 - K_o)\, R/(2 + \frac{4\nu_r E_r R^3}{(3 - 4\nu_r)\,(12(1 + \nu_r)\,E_c I + E_r R^3)}$$

Maximum/minimum radial displacement

$$\frac{u}{R} = \sigma_v\,(I + K_o)\; R^3/(\frac{2}{1 + \nu_r}\,E_r R^3 + 2E_c A R^2 + 2E_c I) \pm \sigma_v\,(I - K_o)\; R^3/(12\,E_c I + \frac{3 - 2\nu_r}{(1 + \nu_r)\,(3 - 4\nu_r)}\,E_r R^3)$$

$$M = 0.068 \times \sigma_v(1 - K_o)R^2 \qquad (9\text{-}17)$$

It is seen that even in this instance, with a relatively rigid lining in a soft rock, the moment is reduced to about 27 percent of the moment that would be obtained in an unsupported ring. Thus, for most lining applications in rock, bending moments are expected to be small.

(2) *Analysis of moments and forces using finite elements computer programs.* Moments and forces in circular and noncircular tunnel linings can be determined using structural finite-element computer programs. Such analyses have the following advantages:

- Variable properties can be given to rock as well as lining elements.
- Irregular boundaries and shapes can be handled.
- Incremental construction loads can be analyzed, including, for example, loads from backfill grouting.
- Two-pass lining interaction can also be analyzed.

In a finite element analysis (FEM) analysis, the lining is divided into beam elements. Hinges can be introduced to simulate structural properties of the lining. Tangential and radial springs are applied at each node to simulate elastic interaction between the lining and the rock. The interface between lining and rock cannot withstand tension; therefore, interface elements may be used or the springs deactivated when tensile stresses occur. The radial and tangential spring stiffnesses, expressed in units of force/

displacement (subgrade reaction coefficient), are estimated from

$$k_r = E_r\ b\ \theta/(1 + \upsilon_r) \qquad (9\text{-}18)$$

$$k_t = k_r\ G/E_r = 0.5\ k_r/(1 + \upsilon_r) \qquad (9\text{-}19)$$

where

k_r and k_t = radial and tangential spring stiffnesses, respectively

G = shear modulus

θ = arc subtended by the beam element (radian)

b = length of tunnel element considered

If a segmental lining is considered, b can be taken as the width of the segment ring. Loads can be applied to any number of nodes, reflecting assumed vertical rock loads acting over part or all of the tunnel width, grouting loads, external loads from groundwater, asymmetric, singular rock loads, internal loads, or any other loads. Loads can be applied in stages, reflecting a sequence of construction. Figure 9-3 shows the FEM model for a two-pass lining system. The initial lining is an unbolted, segmental concrete lining, and the final lining is reinforced cast-in-place concrete with an impervious waterproofing membrane. Rigid links are used to interconnect the two linings at alternate nodes. These links transfer only axial loads and have no flexural stiffness and a minimum of axial deformation. Hinges are introduced at crown, invert, and springlines of the initial lining to represent the joints between the segments.

(3) *Continuum analysis, numerical solutions.*

Continuum analyses (Section 8-4) provide the complete stress state throughout the rock mass and the support structure. These stresses are used to calculate the (axial and shear) forces and the bending moments in the components of the support structure. The forces and moments are provided as a direct output from the computer analyses with no need for an additional calculation on the part of the user. The forces and moments give the designer information on the working load to be applied to the structure and can be used in the reinforced concrete design. Figure 9-4 shows a sample output of moment and force distribution in a lining of a circular tunnel under two different excavation conditions.

(4) *Design of concrete cross section for bending and normal force.* Once bending moment and ring thrust in a lining have been determined, or a lining distortion estimated, based on rock-structure interaction, the lining must be designed to achieve acceptable performance. Since the lining is subjected to combined normal force and bending, the analysis is conveniently carried out using the capacity-interaction curve, also called the moment-thrust diagram. EM 1110-2-2104 should be used to design reinforced concrete linings. The interaction diagram displays the envelope of acceptable combinations of bending moment and axial force in a reinforced or unreinforced concrete member. As shown in Figure 9-5, the allowable moment for low values of thrust increases with the thrust because it reduces the limiting tension across the member section. The maximum allowable moment is reached at the so-called balance point. For higher thrust, compressive stresses reduce the allowable moment. General equations to calculate points of the interaction diagram are shown in EM 1110-2-2104. Each combination of cross-section area and reinforcement results in a unique interaction diagram, and families of curves can be generated for different levels of reinforcement for a given cross section. The equations are easily set up on a computer spreadsheet, or standard structural computer codes can be used. A lining cross section is deemed adequate if the combination of moment and thrust values are within the envelope defined by the interaction diagram. The equations shown in EM 1110-2-2104 are applicable to a tunnel lining of uniform cross section with reinforcement at both interior and exterior faces. Linings with nonuniform cross sections, such as coffered segmental linings, are analyzed using slightly more complex equations, such as those shown in standard structural engineering handbooks, but based on the same principles. Tunnel lining distortion stated as a relative diameter change ($\Delta D/D$) may be derived from computerized rock-structure analyses, from estimates of long-term swelling effects, or may be a nominal distortion derived from past experience. The effect of an assumed distortion can be analyzed using the interaction diagram by converting the distortion to an equivalent bending moment in the lining. For a uniform ring structure, the conversion formula is

$$M = (3EI/R)(\Delta D/D) \qquad (9\text{-}20)$$

In the event that the lining is not properly described as a uniform ring structure, the representation of ring stiffness in this equation ($3EI/R$) should be modified. For example, joints in a segmental lining introduce a reduction in the moment of inertia of the ring that can be approximated by the equation

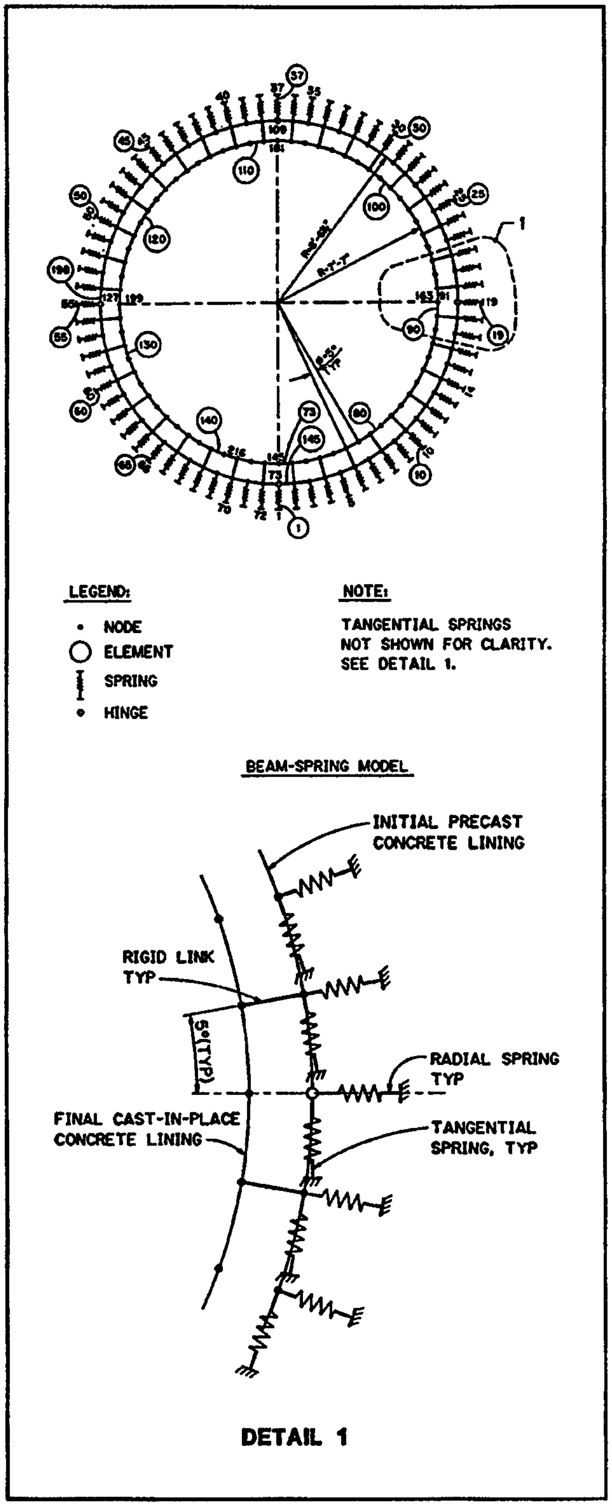


Figure 9-3. Descretization of a two-pass lining system for analysis

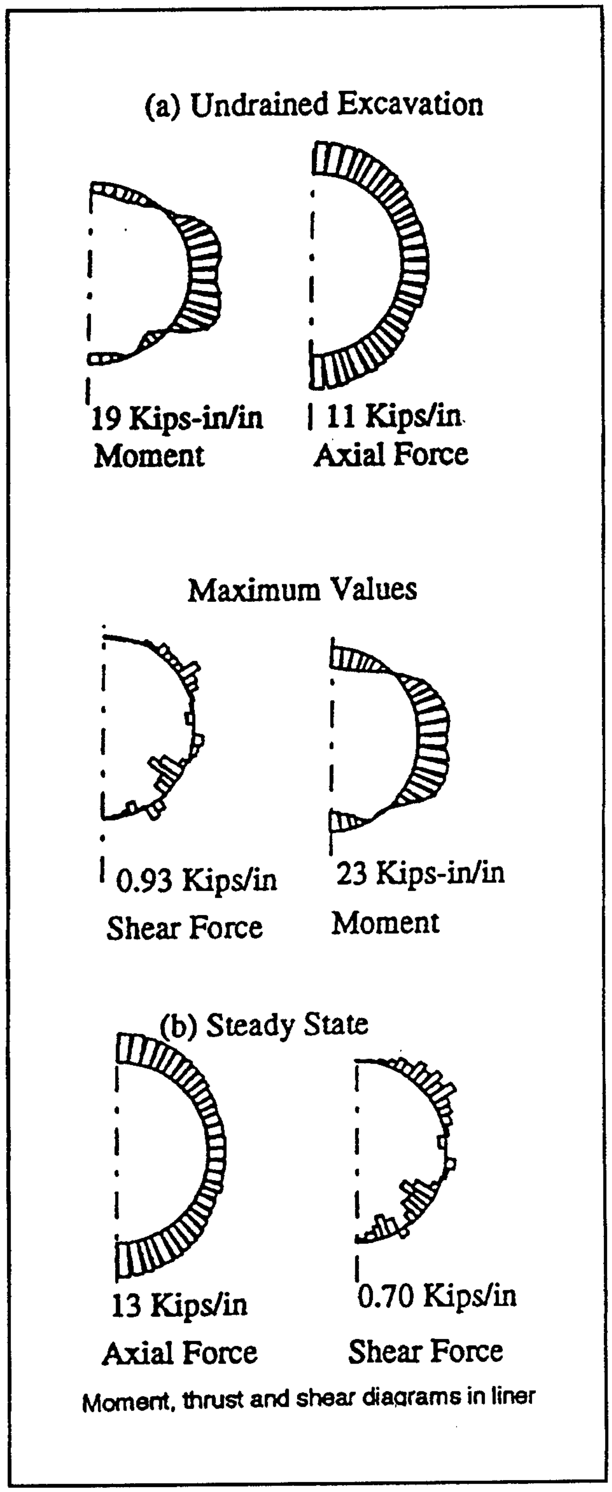


Figure 9-4. Moments and forces in lining shown in Figure 9-3

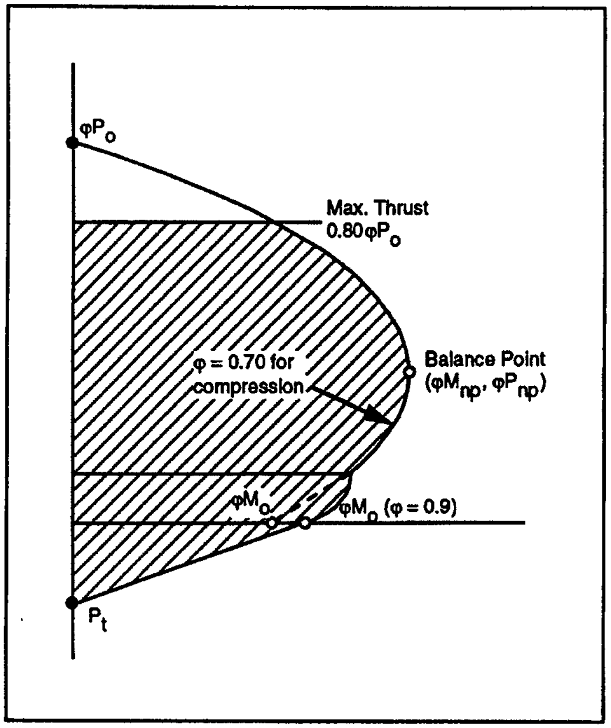


Figure 9-5. Capacity interaction curve

$$I_{eff} = I_j + (4/n)^2 I \qquad (9\text{-}21)$$

where

I = moment of inertia of the lining

I_j = moment of inertia of the joint

n = number of joints in the lining ring where $n > 4$

Alternatively, more rigorous analyses can be performed to determine the effects of joints in the lining. Nonbolted joints would have a greater effect than joints with tensioned bolts. If the estimated lining moment falls outside the envelope of the interaction diagram, the designer may choose to increase the strength of the lining. This may not always be the best option. Increasing the strength of the lining also will increase its rigidity, resulting in a greater moment transferred to the lining. It may be more effective to reduce the rigidity of the lining and thereby the moment in the lining. This can be accomplished by (a) introducing joints or increasing the number of joints and (b) using a thinner concrete section of higher strength and introducing stress relievers or yield hinges at several locations around the ring, where high moments would occur.

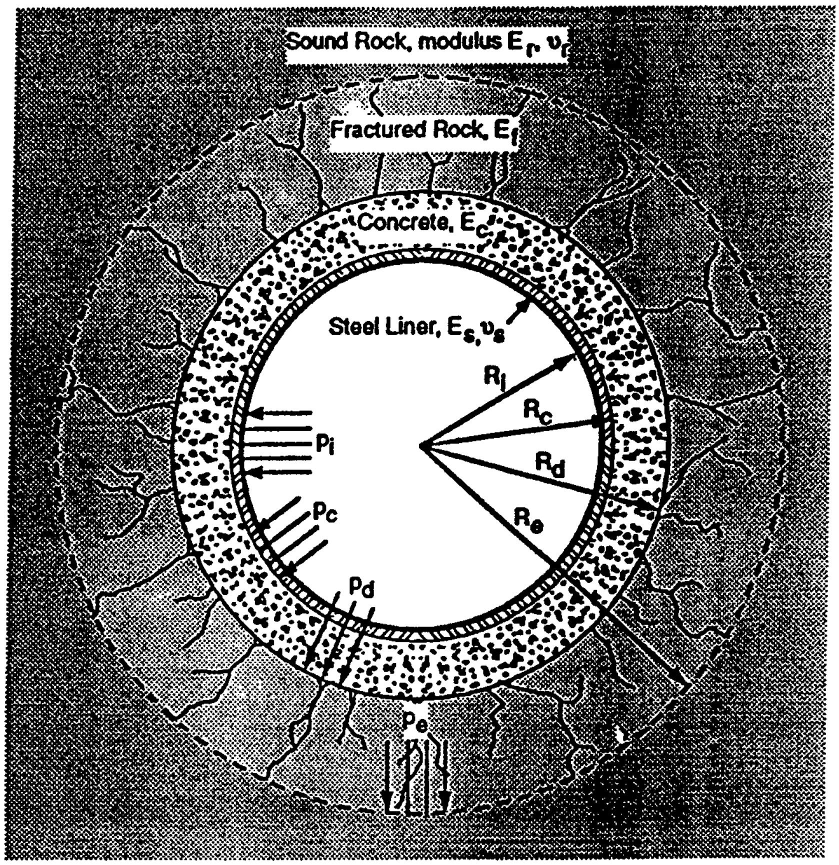
9-5. Design of Permanent Steel Linings

As discussed in Section 9-4, a steel lining is required for pressure tunnels when leakage through cracks in concrete can result in hydrofracturing of the rock or deleterious leakage. Steel linings must be designed for internal as well as for external loads where buckling is critical. When the external load is large, it is often necessary to use external stiffeners. The principles of penstock design apply, and EM 1110-2-3001 provides guidance for the design of steel penstocks. Issues of particular interest for tunnels lined with steel are discussed herein.

a. Design of steel linings for internal pressure. In soft rock, the steel lining should be designed for the net internal pressure, maximum internal pressure minus minimum external formation water pressure. When the rock mass has strength and is confined, the concrete and the rock around the steel pipe can be assumed to participate in carrying the internal pressure. Box 9-3 shows a method of analyzing the interaction between a steel liner, concrete, and a fractured or damaged rock zone, and a sound rock considering the gap between the steel and concrete caused by temperature effects. The extent of the fractured rock zone can vary from little or nothing for a TBM-excavated tunnel to one or more meters in a tunnel excavated by blasting, and the quality of the rock is not well known in advance. Therefore, the steel lining, which must be designed and manufactured before the tunnel is excavated, must be based on conservative design assumption. If the steel pipe is equipped with external stiffeners, the section area of the stiffeners should be included in the analysis for internal pressure.

b. Design considerations for external pressure. Failure of a steel liner due to external water pressure occurs by buckling, which, in most cases, manifests itself by formation of a single lobe parallel to the axis of the tunnel. Buckling occurs at a critical circumferential/ axial stress at which the steel liner becomes unstable and fails in the same way as a slender column. The failure starts at a critical pressure, which depends not only on the thickness of the steel liner but also on the gap between the steel liner and concrete backfill. Realistically, the gap can vary from 0 to 0.001 times the tunnel radius depending on a number of factors, including the effectiveness of contact grouting of voids behind the steel liner. Other factors include the effects of heat of hydration of cement, temperature changes of steel and concrete during construction, and ambient temperature changes due to forced or natural ventilation of the tunnel. For example, the steel liner may reach temperatures 80 °F or more due to ambient air temperature

Box 9-3. Interaction Between Steel Liner, Concrete and Rock



1. Assume concrete and fractured rock ar cracked; then

$p_c R_c = p_d R_d = p_e R_e$;
$p_d = p_c R_c / R_d$; $p_c = p_e R_c / R_e$

2. Steel lining carries pressure $p_i - p_c$ and sustains radial displacement

$\Delta s = (p_i - p_c)\ R_i^2\ (1 - v_s^2) / (tE_s)$

3. $\Delta s = \Delta k + \Delta c + \Delta d + \Delta E$, where
   Δk = radial temperature gap = $C_s \Delta T R_i$ ($C_s = 6.5.10^{-6}/°F$)
   Δc = compression of concrete = $(p_c R_c / E_c) \ln (R_d / R_c)$
   Δd = compression of fractured rock = $(p_c R_c / E_f) \ln (R_e / R_d)$
   Δe = compression of intact rock = $(p_c R_c / E_r)(1 + \upsilon_r)$

4. Hence

$$p_c = (p_i R_i^2 (1 - \upsilon_s^2) / tE_s - C_s \Delta T R_c) / (R_i^2 (1 - \upsilon_s^2) / tE_s + (R_c/E_c) \ln (R_d/R_c) + (R_c/E_f) \ln (R_e/R_d) + R_c (1 + \upsilon_r) / E_r)$$

and the heat of hydration. If the tunnel is dewatered during winter when the water temperature is 34 °F, the resulting difference in temperature would be 46 °F. This temperature difference would produce a gap between the steel liner and concrete backfill equal to 0.0003 times the tunnel radius. Definition of radial gap for the purpose of design should be based on the effects of temperature changes and shrinkage, not on imperfections resulting from inadequate construction. Construction problems must be remedied before the tunnel is put in operation. Stability of the steel liner depends also on the effect of its out-of-roundness. There are practical limitations on shop fabrication and field erection in controlling the out-of-roundness of a steel liner. Large-diameter liners can be fabricated with tolerance of about 0.5 percent of the diameter. In other words, permissible tolerances during fabrication and erection of a liner may permit a 1-percent difference between measured maximum and minimum diameters of its deformed (elliptical) shape. Such flattening of a liner, however, should not be considered in defining the gap used in design formulas. It is common practice, however, to specify internal spider bracing for large-diameter liners, which is adjustable to obtain the required circularity before and during placement of concrete backfill. Spider bracing may also provide support to the liner during contact grouting between the liner and concrete backfill. A steel liner must be designed to resist maximum external water pressure when the tunnel is dewatered for inspection and maintenance. The external water pressure on the steel liner can develop from a variety of sources and may be higher than the vertical distance to the ground surface due to perched aquifers. Even a small amount of water accumulated on the outside of the steel liner can result in buckling when the tunnel is dewatered for inspection or maintenance. Therefore, pressure readings should be taken prior to dewatering when significant groundwater pressure is expected. Design of thick steel liners for large diameter tunnels is subject to practical and economic limitations. Nominal thickness liners, however, have been used in large-diameter tunnels with the addition of an external drainage system consisting of steel collector pipes with drains embedded in concrete backfill. The drains are short, small-diameter pipes connecting the radial gap between the steel liner and concrete with the collectors. The collectors run parallel to the axis of the tunnel and discharge into a sump inside the power house. Control valves should be provided at the end of the collectors and closed during tunnel operations to prevent unnecessary, continuous drainage and to preclude potential clogging of the drains. The valves should be opened before dewatering of the tunnel for scheduled maintenance and inspection to allow drainage.

c. Design of steel liners without stiffeners. Analytical methods have been developed by Amstutz (1970), Jacobsen (1974), and Vaughan (1956) for determination of critical buckling pressures for cylindrical steel liners without stiffeners. Computer solutions by Moore (1960) and by MathCad have also been developed. The designer must be aware that the different theoretical solutions produce different results. It is therefore prudent to perform more than one type of analyses to determine safe critical and allowable buckling pressures. Following are discussions of the various analytical methods.

(1) *Amstutz's analysis.* Steel liner buckling begins when the external water pressure reaches a critical value. Due to low resistance to bending, the steel liner is flattened and separates from the surrounding concrete. The failure involves formation of a single lobe parallel to the axis of the tunnel. The shape of lobe due to deformation and elastic shortening of the steel liner wall is shown in Figure 9-6.

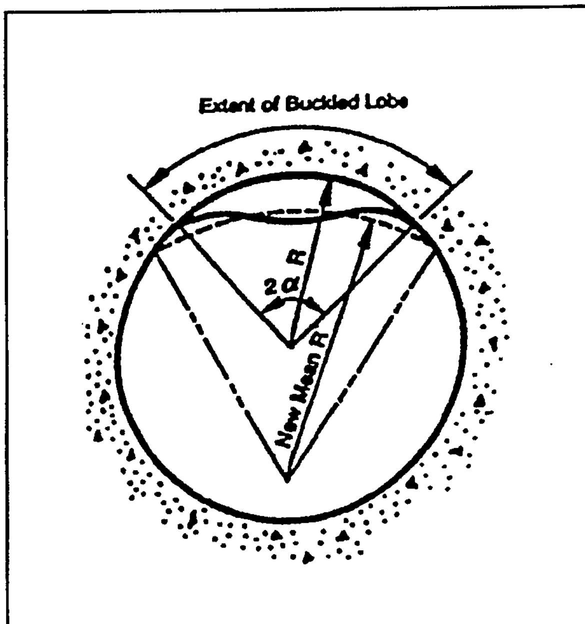


Figure 9-6. Buckling, single lobe

The equations for determining the circumferential stress in the steel-liner wall and corresponding critical external pressure are:

$$\frac{\sigma_N - \sigma_v}{\sigma_F^* - \sigma_N}\left[\left(\frac{r}{i}\right)\frac{\sqrt{\sigma_N}}{E^*}\right]^3 \cong 1.73\left(\frac{r}{e}\right)\left[1 - 0.225\left(\frac{r}{e}\right)\frac{\sigma_F^* - \sigma_N}{E^*}\right] \tag{9-22}$$

$$P_{cr} \cong \left(\frac{F}{r}\right)\sigma_N\left(1 - 0.175\left(\frac{r}{e}\right)\frac{\sigma_F^* - \sigma_N}{E^*}\right) \tag{9-23}$$

where

$i = t/\sqrt{12}$, $e = t/2$, $F = t$

$\sigma_v = -(k/r)E^*$

k/r = gap ratio between steel and concrete = γ

r = tunnel liner radius

t = plate thickness

E = modulus of elasticity

$E^* = E/(1-v^2)$

σ_y = yield strength

σ_N = circumferential/axial stress in plate liner

$\mu = 1.5\text{-}0.5[1/(1+0.002\ E/\sigma_y)]^2$

$\sigma_F^* = \mu\sigma_y\sqrt{1-v+v^2}$

v = Poisson's Ratio

In general, buckling of a liner begins at a circumferential/axial stress (σ_N) substantially lower than the yield stress of the material except in liners with very small gap ratios and in very thick linings. In such cases σ_N approaches the yield stress. The modulus of elasticity (E) is assumed constant in Amstutz's analysis. To simplify the analysis and to reduce the number of unknown variables, Amstutz introduced a number of coefficients that remain constant and do not affect the results of calculations. These coefficients are dependent on the value of ε, an expression for the inward deformation of the liner at any point, see Figure 9-7. Amstutz indicates that the acceptable range for values of ε is $5<\varepsilon<20$. Others contend that the ε dependent coefficients are more acceptable in the range $10<\varepsilon<20$, as depicted by the flatter portions of the curves shown in Figure 9-7. According to Amstutz, axial stress (σ_N) must be determined in conjunction with the corresponding value of ε. Thus, obtained results may be considered satisfactory providing $\sigma_N<0.8\sigma_y$. Figure 9-8 shows curves based on Amstutz equations (after Moore 1960). Box 9-4 is a MathCad application of Amstutz's equations.

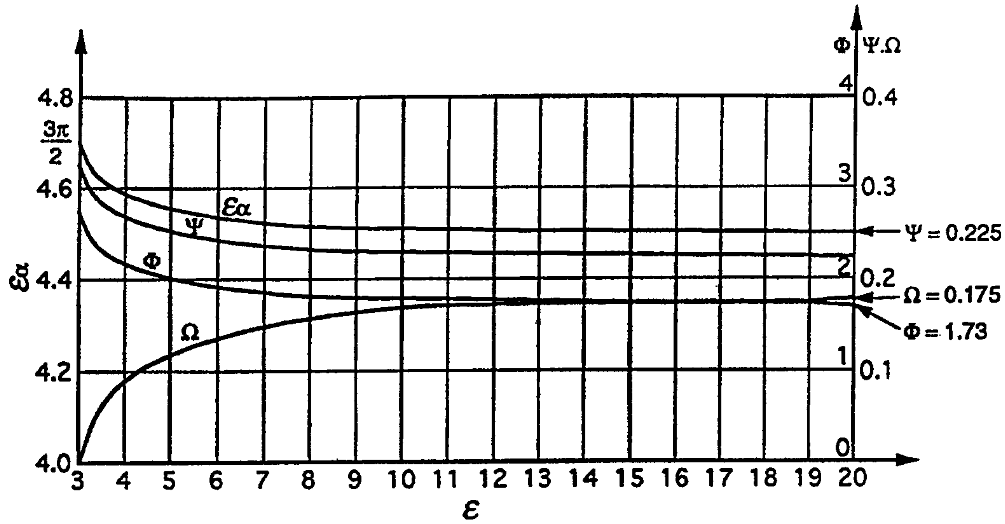
(2) *Jacobsen's analysis.* Determination of the critical external buckling pressure for cylindrical steel liners without stiffeners using Jacobsen's method requires solution of three simultaneous nonlinear equations with three unknowns. It is, however, a preferred method of design since, in most cases, it produces lower critical allowable buckling pressures than Amstutz's method. A solution of Jacobsen equations using MathCad is shown in Box 9-5.

The three equations with three unknowns α, β, and p in Jacobsen's analysis are:

$$r/t = \frac{\sqrt{[(9\pi^2/4\ \beta^2) - 1]\ [\pi - \alpha + \beta\ (\sin\alpha / \sin\beta)^2]}}{12\ (\sin\alpha/\sin\beta)^3\ [\alpha - (\pi\Delta/r) - \beta(\sin\alpha/\sin\beta)\ [1 + \tan^2(\alpha - \beta)/4]]} \tag{9-24}$$

$$p/E^* = \frac{(9/4)\ (\pi/\beta)^2 - 1}{12\ (r/t)^3\ (\sin\alpha/\sin\beta)^3} \tag{9-25}$$

$$\sigma_y/E^* = (t/2r)\ [1 - (\sin\beta/\sin\alpha)] + (pr\sin\alpha/E^*\,t\sin\beta)\left[1 + \frac{4\beta\ r\sin\alpha\tan(\alpha - \beta)}{\pi\ t\sin\beta}\right] \tag{9-26}$$



Note: At ε = 2, α = 180° (εα = 360°) and Φ and Ψ → ∞

ε	εα°	α°	tan εα	tan α	ε tan α	cos (εα)	sin (εα)	sin α	εα	β (25)	γ (29)	δ (35)	Φ (39)	Ψ (40)	Ω (48)
3	270°00'	90°00'	∞	∞	∞	0	-1.00000	-1.0000	4.71239	-2.6667	28.3	8.00	2.88	0.331	0
4	263°37',2	65°54',3	8.9446	2.2360	8.9440	-0.11112	-0.99381	0.91287	4.60104	-1.8095	32.7	16.67	2.21	0.271	0.100
5	261°11',6	52°14',3	6.4550	1.2910	6.4550	-0.15310	-0.98821	0.79056	4.55868	-1.3933	38.7	27.67	2.00	0.251	0.133
10	258°19',7	25°50'	4.8409	0.48413	4.8413	-0.20231	-0.97932	0.43575	4.50868	-0.6650	71.4	119.03	1.78	0.226	0.168
20	257°40',2	12°53'	4.5749	0.22873	4.5746	-0.21357	-0.97693	0.22297	4.49719	-0.3286	143.4	484.2	1.73	0.225	0.175

Figure 9-7. Amstutz coefficients as functions of "ε"

where

α = one-half the angle subtended to the center of the cylindrical shell by the buckled lobe

β = one-half the angle subtended by the new mean radius through the half waves of the buckled lobe

p = critical external buckling pressure, psi

Δ/r = gap ratio, for gap between steel and concrete

r = tunnel liner internal radius, in.

σ_y = yield stress of liner, psi

t = liner plate thickness, in.

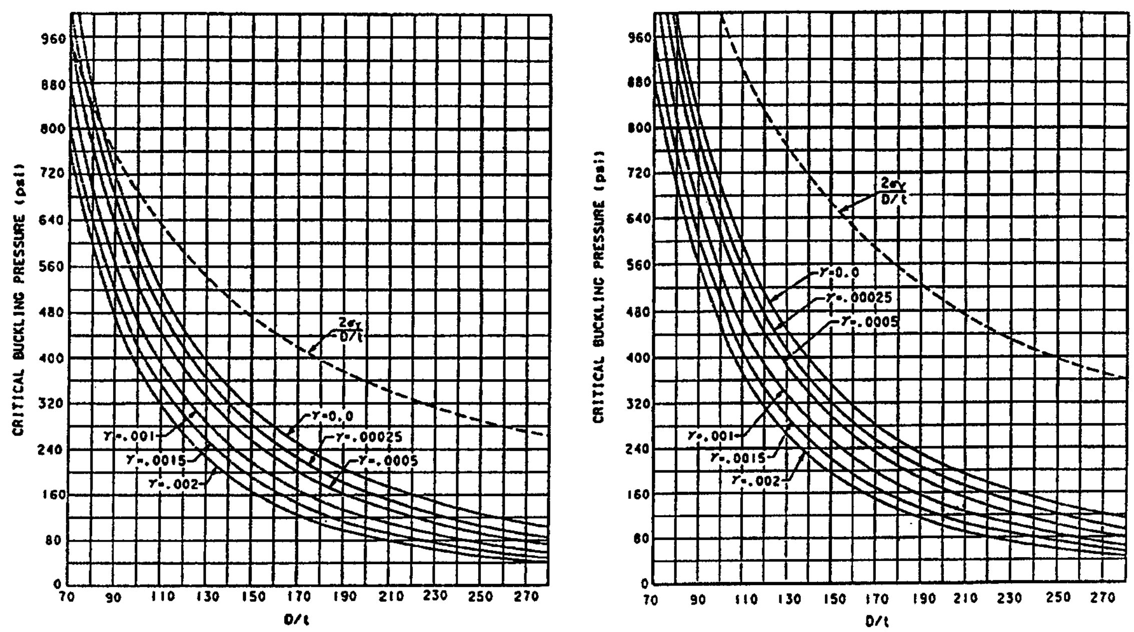


Figure 9-8. Curves based on Amstutz equations by E. T. Moore

E^* = modified modulus of elasticity, $E/(1-\nu_2)$

ν = Poisson's Ratio for steel

Curves based on Jacobsen's equations for the two different steel types are shown on Figure 9-9.

(3) *Vaughan's analysis.* Vaughan's mathematical equation for determination of the critical external buckling pressure is based on work by Bryan and the theory of elastic stability of thin shells by Timoshenko (1936). The failure of the liner due to buckling is not based on the assumption of a single lobe; instead, it is based on distortion of the liner represented by a number of waves as shown in Figure 9-10.

$$\left[\frac{\sigma_y - \sigma_{cr}}{2E^*} + \frac{6\sigma_{cr}}{\sigma_{y} - \sigma_{cr}}\left(\frac{y_o}{R} + \frac{\sigma_{cr}}{E^*}\right)\right] \times \qquad (9\text{-}27)$$

$$\frac{R^2}{T^2} - \frac{R}{T} + \frac{\sigma_y - \sigma_{cr}}{24\sigma_{cr}} = 0$$

where

σ_y = yield stress of liner, psi

σ_{cr} = critical stress

$E^* = E/(1-\nu^2)$

y_o = gap between steel and concrete

R = tunnel liner radius

T = plate thickness

Box 9-6 is a MathCad example of the application of Vaughan's analysis. Vaughan provides a family of curves (Figure 9-11) for estimating approximate critical pressures. These curves are for steel with σ_y = 40,000 psi with various values of y_o/R. It is noted that approximate pressure values obtained from these curves do not include a safety factor.

Box 9-4. MathCad Application of Amstutz's Equations

Liner thickness t = 0.50 in. **ASTM A 516 - 70**

$t := 0.50 \quad F := 0.50 \quad r := 90 \quad k := 0.027 \quad \frac{k}{r} = 3 \cdot 10^{-4}$

$E := 30 \cdot 10^6 \quad \sigma_F := 38 \cdot 10^3 \quad v := 0.30 \quad \frac{t}{2} = 0.25 \quad 2 \cdot \frac{r}{t} = 360$

$\frac{30 \cdot 10^6}{(1 - v^2)} = 3.297 \cdot 10^7 \quad E_m := 3.297 \cdot 10^7 \quad \frac{t}{\sqrt{12}} = 0.144 \quad i := 0.17 \quad \frac{r}{i} = 529.412$

$-\frac{k}{r} \cdot E_m = -9.891 \cdot 10^3 \quad \sigma_v := -9.891 \cdot 10^3$

$$1.5 - 0.5 \cdot \left[\left[\frac{1}{1 + 0.002 \cdot \frac{E}{\sigma_F}}\right]^2\right] = 1.425 \qquad \mu := 1.425$$

$$\frac{\mu \cdot \sigma_F}{\sqrt{1 - v + v^2}} = 6.092 \cdot 10^4 \qquad \sigma^m := 6.092 \cdot 10^4$$

$$\sigma_N := 12 \cdot 10^3$$

$$a := \sqrt{\left[\left(\frac{\sigma_N - \sigma_v}{\sigma_m - \sigma_N}\right) \cdot \left[\left(\frac{r}{i}\right) \cdot \sqrt{\frac{\sigma_N}{E_m}}\right]^3 - \left[1 - 0.225 \cdot \frac{2 \cdot r}{t} \cdot \left(\frac{\sigma_m - \sigma_N}{E_m}\right)\right] \cdot 1.73 \cdot \frac{2 \cdot r}{t} \cdot \sigma_N\right]}$$

$a = 1.294 \cdot 10^4$

$t := 0.50 \quad F := 0.50 \quad r := 90 \quad \sigma_N := 1.294 \cdot 10^4 \quad i := 0.17 \quad E_m := 3.297 \cdot 10^7 \quad \sigma_m := 6.092 \cdot 10^4$

$$\left(\frac{F}{r}\right) \cdot \sigma_N \cdot \left[1 - 0.175 \cdot \left(\frac{2 \cdot r}{t}\right) \cdot \left(\frac{\sigma_m - \sigma_N}{E_m}\right)\right] = 65.298$$

External pressures:

Critical buckling pressure = 65 psi
Allowable buckling pressure = 43 psi **(Safety Factor = 1.5)**

d. Design examples. There is no one single procedure recommended for analysis of steel liners subjected to external buckling pressures. Available analyses based on various theories produce different results. The results depend, in particular, on basic assumptions used in derivation of the formulas. It is the responsibility of the designer to recognize the limitations of the various design procedures. Use of more than one procedure is recommended to compare and verify final results and to define safe allowable buckling pressures. Most of the steel liner buckling problems can best be solved with MathCad computer applications. Table 9-2 shows the results of MathCad applications in defining allowable buckling pressures for a 90-in. radius (ASTM A 516-70) steel liner with varying plate thicknesses: 1/2, 5/8, 3/4, 7/8, and 1.0 in. Amstutz's and Jacobsen's analyses are based on the assumption of a single-lobe buckling failure. Vaughan's analysis is based on multiple-waves failure that produces much higher

Box 9-5. MathCad Application of Jacobsen's Equations

Liner thickness t = 0.50 in. **ASTM A 516-70**

$t := 0.50 \qquad r := 90 \qquad \Delta := 0.027 \qquad \frac{\Delta}{r} = 3 \cdot 10^{-4}$

$E := 30 \cdot 10^{6} \qquad \sigma_y := 38 \cdot 10^{3} \qquad v := 0.30$

$\frac{30 \cdot 10^{6}}{(1 - v^{2})} = 3.297 \cdot 10^{7} \qquad E_m := 3.296 \cdot 10^{7}$

Guesses $\alpha := 0.35 \qquad \beta := 0.30 \qquad p := 40$

Given

$$\frac{r}{t} = \sqrt{\left[\frac{\left[\left[\frac{9 \cdot \pi^2}{4 \cdot (\beta)^2}\right] - 1\right] \cdot \left[\pi - (\alpha) + (\beta) \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^2\right]}{12 \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^3 \cdot \left[(\alpha) - \left(\frac{\pi \cdot \Delta}{r}\right) - (\beta) \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right) \cdot \left[1 + \left[\frac{\tan((\alpha) - (\beta))}{4}\right] \cdot \tan((\alpha) - (\beta))\right]\right]}\right]}$$

$$\frac{p}{E_m} = \frac{\left(\frac{9}{4}\right) \cdot \left[\frac{\pi}{(\beta)}\right]^2 - 1}{12 \cdot \left(\frac{r}{t}\right)^3 \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^3}$$

$$\frac{\sigma_y}{E_m} = \left(\frac{t}{2 \cdot r}\right) \cdot \left[1 - \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)\right] + \frac{p \cdot r \cdot \sin(\alpha)}{E_m \cdot t \cdot \sin(\beta)} \cdot \left[1 + \frac{4 \cdot (\beta) \cdot r \cdot \sin(\alpha) \cdot \tan((\alpha) - (\beta))}{\pi \cdot t \cdot \sin(\beta)}\right]$$

$$\text{minerr}(\alpha, \beta, p) = \begin{pmatrix} 0.409 \\ 0.37 \\ 51.321 \end{pmatrix}$$

External pressures:

Critical buckling pressure = 51 psi
Allowable buckling pressure = 34 psi **(Safety Factor = 1.5)**

Table 9-2
Allowable Buckling Pressures for a 90-in.-diam. Steel Liner Without Stiffeners

		Plat Thicknesses, in., ASTM A516-70				
Analyses/ Formulas	Safety Factor	1/2	5/8	3/4	7/8	1.0
		Allowable Buckling Pressures, psi				
Amstutz	1.5	65	82	119	160	205
Jacobsen	1.5	51	65	116	153	173
Vaughan	1.5	97	135	175	217	260

allowable buckling pressures. Based on experience, most of the buckling failures involve formation of a single lobe; therefore, use of the Amstutz's and Jacobsen's equations to determine allowable buckling pressures is recommended.

e. Design of steel liners with stiffeners.

(1) *Design considerations.* Use of external circumferential stiffeners should be considered when the thickness of an unstiffened liner designed for external pressure exceeds the thickness of the liner required by the design for internal pressure. Final design should be based on economic considerations of the following three available options that would satisfy the design requirements for the external pressure: (a) increasing the thickness of the liner, (b) adding external stiffeners to the liner using the thickness required for internal pressure, and (c) increasing the

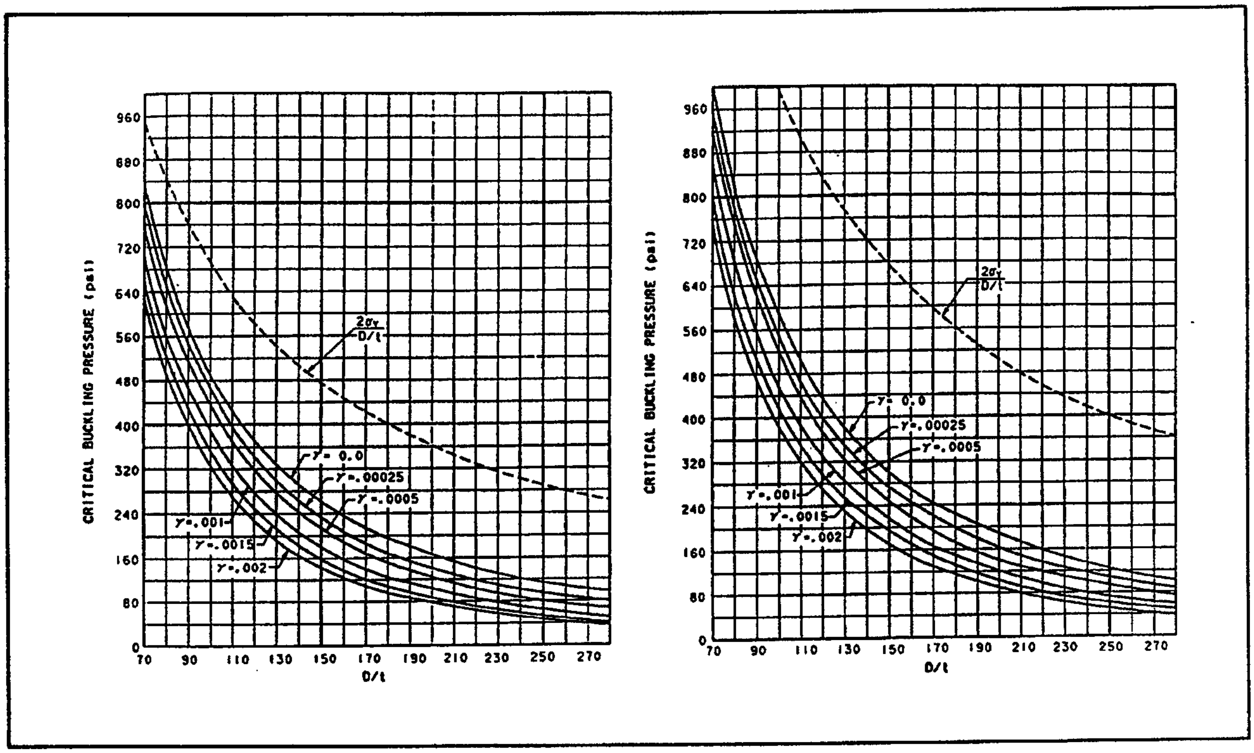


Figure 9-9. Curves based on Jacobsen equations by E. T. Moore

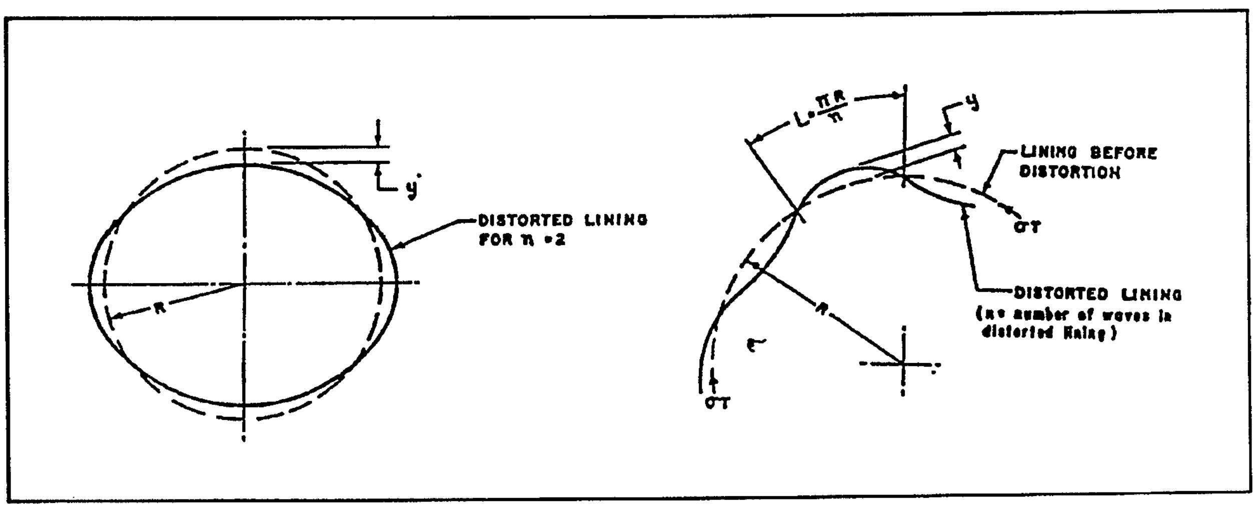


Figure 9-10. Vaughan's buckling patterns - multiple waves

thickness of the liner and adding external stiffeners. The economic comparison between stiffened and unstiffened linings must also consider the considerable cost of additional welding, the cost of additional tunnel excavation required to provide space for the stiffeners, and the additional cost of concrete placement. Several analytical methods are available for design of steel liners with stiffeners. The analyses by von Mises and Donnell are based on distortion of a liner represented by a number of waves, frequently referred to as rotary-symmetric buckling. Analyses by E. Amstutz and by S. Jacobsen are based on a

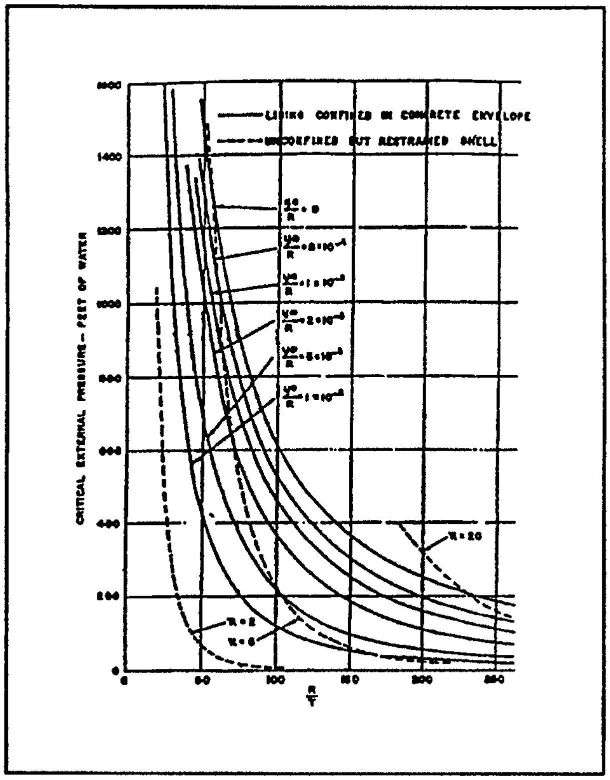


Figure 9-11. Vaughan's curves for yield stress 40,000 psi

single-lobe buckling. Roark's formula is also used. In the single-lobe buckling of liners with stiffeners, the value of ε, an expression for inward deformation of the liner, is generally less than 3; therefore, the corresponding subtended angle 2α is greater than 180° (see Figure 9-7). Since the Amstutz analysis is limited to buckling with ε greater than 3, i.e., 2α less than 180°, it is not applicable to steel liners with stiffeners. For this reason, only Jacobsen's analysis of a single-lobe failure of a stiffened liner is included in this manual, and the Amstutz analysis is not recommended.

(2) *Von Mises's analysis.* Von Mises's equation is based on rotary-symmetric buckling involving formation of a number of waves (n), the approximate number of which can be determined by a formula based on Winderburg and Trilling (1934). A graph for collapse of a free tube derived from von Mises's formula can be helpful in determining buckling of a tube. It is noted that similar equations and graphs for buckling of a free tube have been developed by Timoshenko (1936) and Flügge (1960). Von Mises's equation for determination of critical buckling pressure is:

$$P_{cr} = \frac{E\left(\frac{t}{r}\right)}{1 - \nu^2}\left[\frac{1 - \nu^2}{(n^2 - 1)\left(\frac{n^2 L^2}{\pi^2 r^2} + 1\right)^2}\right] + \frac{E\left(\frac{t}{r}\right)}{12\,(1 - \nu^2)}\left[n^2 - 1 + \frac{2n^2 - 1 - \nu}{\frac{n^2 L^2}{\pi^2 r^2}} - 1\right] \tag{9-28}$$

where

P_{cr} = collapsing pressure psi, for FS = 1.0

r = radius to neutral axis of the liner

ν = Poisson's Ratio

E = modulus of elasticity, psi

t = thickness of the liner, in.

L = distance between the stiffeners, i.e., center-to-center of stiffeners, in.

n = number of waves (lobes) in the complete circumference at collapse

Figure 9-12 shows in graphic form a relationship between critical pressure, the ratio of L/r and the number of waves at the time of the liner collapse. This graph can be used for an approximate estimate of the buckling pressure and the number of waves of a free tube. The number of waves n is an integer number, and it is not an independent variable. It can be determined by trial-and-error substitution starting with an estimated value based on a graph. For practical purposes, $6 \leq n \geq 14$. The number of waves n can also be estimated from the equation by Winderburg and Trilling (1934). The number of waves in the rotary-symmetric buckling equations can also be estimated from the graph shown in Figure 9-12.

(3) *Winderburg's and Trilling's equation.*

Winderburg and Trilling's equation for determination of number of waves n in the complete circumference of the steel liner at collapse is:

$$n = \sqrt[4]{\frac{\dfrac{3\pi^2}{4\sqrt{(1 - \nu^2)}}}{\left(\dfrac{L}{D}\right)^2\left(\dfrac{t}{D}\right)}} \tag{9-29}$$

Box 9-6. MathCad Application of Vaughan's Equations

Liner thickness t = 0.50 in. **ASTM A 516-70**

$T := 0.50$ $R := 90$ $\sigma_y := 38 \cdot 10^3$ $y_o := 0.027$ $\frac{y_o}{R} = 3 \cdot 10^{-4}$

$\nu := 0.3$ $\frac{30 \cdot 10^6}{1 - \nu^2} = 3.297 \cdot 10^7$ $E_m := 3.296 \cdot 10^7$ $\sigma_{cr} := 12 \cdot 10^3$

$$a := \sqrt{\left[\left[\frac{\sigma_y - \sigma_{cr}}{2 \cdot E_m} + \frac{6 \cdot \sigma_{cr}}{\sigma_y - \sigma_{cr}} \cdot \left(\frac{y_o}{R} + \frac{\sigma_{cr}}{E_m}\right)\right] \cdot \frac{R^2}{T^2} - \frac{R}{T} + \left(\frac{\sigma_y - \sigma_{cr}}{24 \cdot \sigma_{cr}}\right), \sigma_{cr}\right]}$$

$a = 1.901 \cdot 10^4$ $\sigma_{cr} := 1.901 \cdot 10^4$

$T := 0.50$ $R := 90$ $\sigma_{cr} := 1.901 \cdot 10^4$ $\sigma_m := 6.092 \cdot 10^4$ $E_m := 3.297 \cdot 10^7$

$$\left(\frac{T}{R}\right) \cdot \sigma_{cr} \cdot \left[1 - 0.175 \cdot \left(\frac{2 \cdot R}{T} \cdot \frac{\sigma_m - \sigma_{cr}}{E_m}\right)\right] = 97.153$$

External pressures:

Critical buckling pressure = 97 psi
Allowable buckling pressure = 65 psi **(Safety Factor = 1.5)**

The above equation determines number of waves n for any Poisson's Ratio. For $\nu = 0.3$, however, the above equation reduces to:

$$n = \sqrt[4]{\frac{7.061}{\left(\frac{L}{D}\right)^2 \left(\frac{t}{D}\right)}} \tag{9-30}$$

Figure 9-13 shows the relationship between n, length/diameter ratio, and thickness/diameter ratio using this equation.

(4) *Donnell's analysis.*

Donnell's equation for rotary-symmetric buckling is:

$$Pcr = \frac{EI_s}{R^3}\left[\frac{(n^2 + \lambda^2)}{N^2}\right] + \frac{EI_s}{R}\left[\frac{\lambda^2}{n^2 (n^2 + \lambda^2)^2}\right] \tag{9-31}$$

where

P_{cr} = collapsing pressure, for FS = 1.0

R = shell radius, in.

I_s = shell bending stiffness, $t^3/12(1 - \nu^2)$

ν = Poisson's Ratio

E = modulus of elasticity

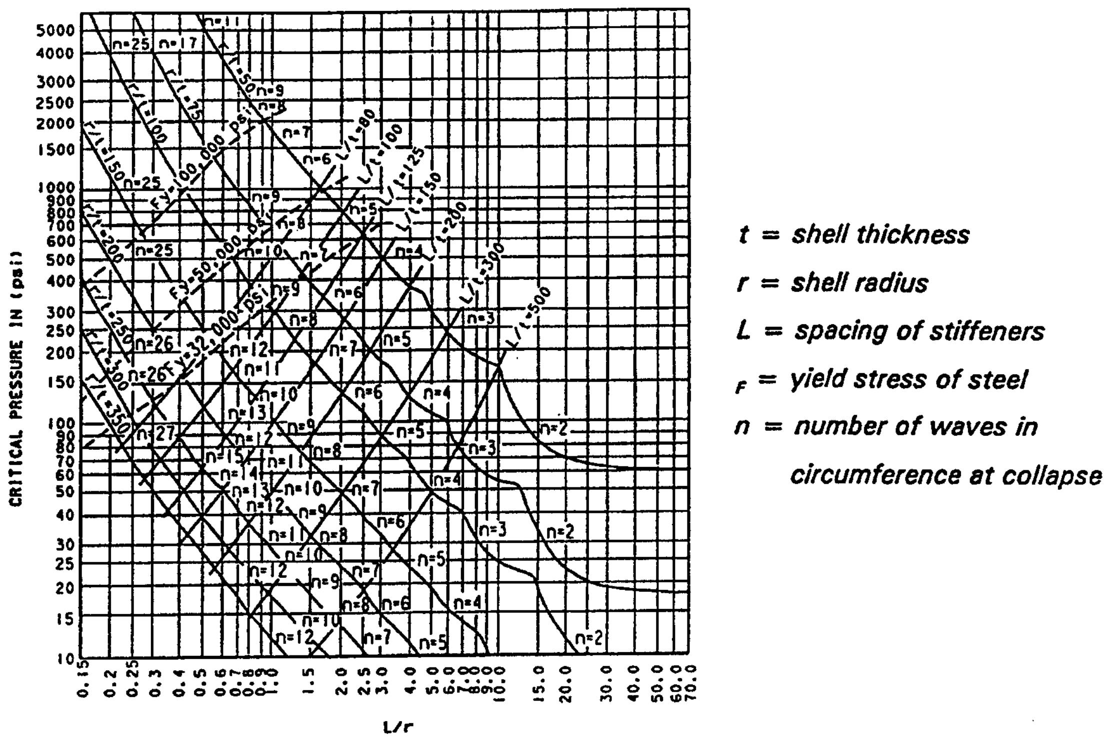


Figure 9-12. Collapse of a free tube (R. von Mises)

t = shell thickness

$\lambda = \pi R/L$

L = length of tube between the stiffeners

n = number of waves (lobes) in the complete circumference at collapse

(5) *Roark's formula.* When compared with other analyses, Roark's formula produces lower, safer, critical buckling pressures. Roark's formula for critical buckling is:

$$P_{cr} = \frac{0.807\ E_s\ t^2}{L_1\ R_1} \sqrt[4]{\left(\frac{1}{1 - v^2}\right)^3 \frac{t^2}{R_1^2}} \qquad (9\text{-}32)$$

where

E = modulus of elasticity of steel

t = thickness of the liner

R_1 = radius to the inside of the liner

v = Poisson's ratio for steel

L_1 = spacing of anchors (stiffeners)

(6) *Jacobsen's equations.* Jacobsen's analysis of steel liners with external stiffeners is similar to that without stiffeners, except that the stiffeners are included in computing the total moment of inertia, i.e., moment of inertia of the stiffener with contributing width of the shell equal to $1.57\ \sqrt{rt} + t_s$. As in the case of unstiffened liners, the analysis of liners with stiffeners is based on the assumption of a single-lobe failure. The three simultaneous equations with three unknowns α, β, and p are:

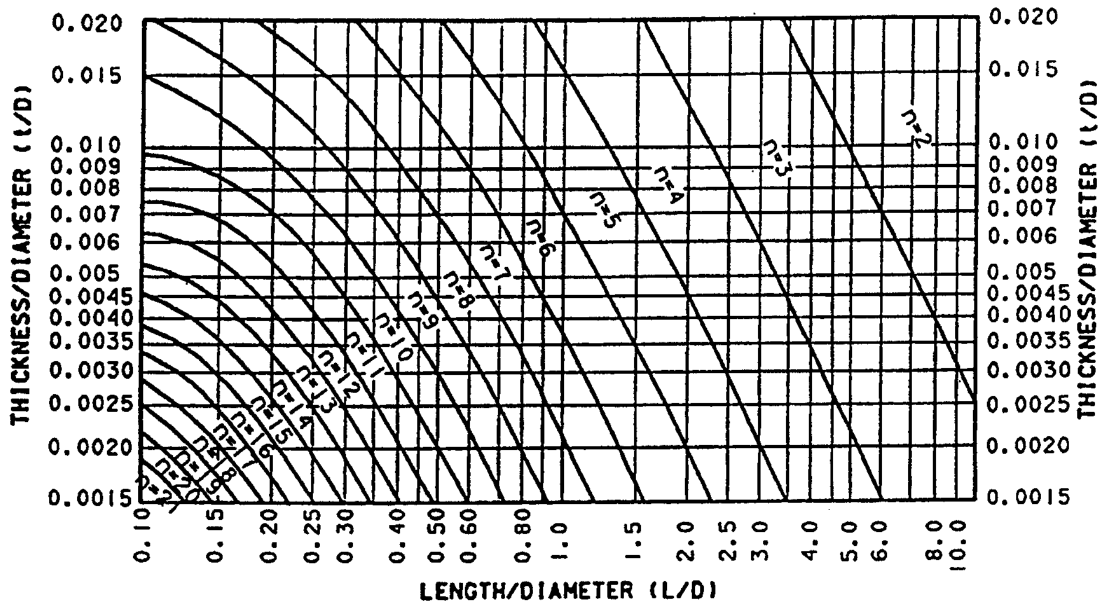


Figure 9-13. Estimation of *n* (Winderburg and Trilling)

$$r/\sqrt{(12J/F)} = \left\{ \frac{[(9\pi^2/4\beta^2) - 1]\ [\pi - \alpha + \beta\,(\sin\alpha/\sin\beta)^2]}{12(\sin\alpha/\sin\beta)^3\ [\alpha - (\pi\Delta/r) - \beta\,(\sin\alpha/\sin\beta)\ [1 + \tan^2(\alpha - \beta)/4]]} \right\}^{1/2} \tag{9-33}$$

$$(p/EF) = \frac{[(9\pi^2/4\beta^2) - 1]}{(r^3 \sin^3\alpha)/[(J/F)\sqrt{\sin^3\beta}\,]} \tag{9-34}$$

$$\sigma_y/l = \frac{h}{r}\left(1 - \frac{\sin\beta}{\sin\alpha}\right) + \frac{p\ r\ \sin\alpha}{EF\ \sin\beta}\left[1 + \frac{8\ \alpha\ hr\ \sin\alpha\ \tan(\alpha - \beta)}{\pi\ \sin\beta\,12J/F}\right] \tag{9-35}$$

where

α = one-half the angle subtended to the center of the cylindrical shell by the buckled lobe

β = one-half the angle subtended by the new mean radius through the half waves of the buckled lobe

p = critical external buckling pressure

J = moment of inertia of the stiffener and contributing width of the shell

F = cross-sectional area of the stiffener and the pipe shell between the stiffeners

h = distance from neutral axis of stiffener to the outer edge of the stiffener

r = radius to neutral axis of the stiffener

σ = yield stress of the liner/stiffener

E = modulus of elasticity of liner/stiffener

Δ/r = gap ratio, i.e., gap/liner radius

Box 9-7 shows a MathCad application of Jacobsen's equation.

(7) *Examples.* Von Mises's and Donnell's equations for rotary-symmetric buckling can best be solved by MathCad application. MathCad application does not require a prior estimate of number of waves n in the circumference of the steel liner at collapse. Instead, a range of n values is defined at the beginning of either equation and, as a result, MathCad produces a range of values for critical pressures corresponding to the assumed n values. Critical pressures versus number of waves are plotted in graphic form. The lowest buckling pressure for each equation is readily determined from the table produced by

Box 9-7. Liner with Stiffeners-Jacobsen Equations

Liner thickness t = 0.500 in.
Stiffeners: 7/8" x 6" @ 48 in. on centers

$r := 90 \qquad J := 44.62 \qquad F := 29.25 \qquad E := 30 \cdot 10^6$

$\Delta := 0.027 \qquad \frac{\Delta}{r} = 3 \cdot 10^{-4} \qquad h := 4.69 \qquad \sigma_y := 38 \cdot 10^3$

Guesses $\quad \alpha := 1.8 \qquad \beta := 1.8 \qquad p := 125$

Given

$$\frac{r}{\sqrt{\left[\frac{12 \cdot J}{F}\right)}} = \sqrt{\left[\frac{\left[\left[\frac{9 \cdot \pi^2}{4 \cdot (\beta)^2}\right] - 1\right] \cdot \left[\pi - (\alpha) + (\beta) \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^2\right]}{12 \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^3 \cdot \left[(\alpha) - \left(\frac{\pi \cdot \Delta}{r}\right) - (\beta) \cdot \left(\frac{\sin(\alpha)}{\sin(\beta)}\right) \cdot \left[1 + \frac{\tan((\alpha) - (\beta)))^2}{4}\right]\right]}\right]}$$

$$\left(\frac{p}{E \cdot F}\right) \cdot \sqrt{\left(\frac{12 \cdot J}{F}\right)} = \frac{\left[\left[\frac{9 \cdot \pi^2}{4 \cdot (\beta)^2}\right] - 1\right]}{\left[\frac{r^3 \cdot \sin(\alpha)^3}{\frac{J}{F} \cdot \sqrt{\frac{12 \cdot J}{F}} \cdot \sin(\beta)^3}\right]}$$

$$\frac{\sigma_y}{E} = \frac{h \cdot \sqrt{\frac{12 \cdot J}{F}}}{R \cdot \sqrt{\left(\frac{12 \cdot J}{F}\right)}} \cdot \left(1 - \frac{\sin(\beta)}{\sin(\alpha)}\right) + \frac{p \cdot \sqrt{\frac{12 \cdot J}{F}} \cdot r \cdot \sin(\alpha)}{E \cdot F \cdot \sqrt{\left(\frac{12 \cdot J}{F}\right)} \cdot \sin(\beta)} \cdot \left[1 + \frac{8 \cdot (\beta) \cdot h \cdot r \cdot \sin(\alpha) \cdot \tan((\alpha) - (\beta))}{\pi \cdot \sqrt{\frac{12 \cdot J}{F}} \cdot \sqrt{\left(\frac{12 \cdot J}{F}\right)} \cdot \sin(\beta)}\right]$$

$p < 130$

$$minerr(\alpha, \beta, p) = \begin{pmatrix} 1.8 \\ 1.8 \\ 126.027 \end{pmatrix}$$

External pressures:

Pcr. (critical buckling pressure) = 126 psi
Pall. (allowable buckling pressure) = 84 psi

MathCad computations. Design examples for determination of critical buckling pressures are included in Boxes 9-8, 9-9, and 9-10. Number of waves in the complete circumference at the collapse of the liner can best be determined with MathCad computer applications as shown in Box 9-11. Table 9-3 below shows that allowable buckling pressures differ depending on the analyses used for computations of such pressures. A designer must be cognizant of such differences as well as the design limitations of various procedures to determine safe allowable buckling pressures for a specific design. An adequate safety factor must be used to obtain safe allowable pressures, depending on a specific analysis and the mode of buckling failure assumed in the analysis.

f. Transitions between steel and concrete lining. In partially steel-lined tunnels, the transition between the steel-lined and the concrete-lined portions of the tunnel requires special design features. Seepage rings are usually installed at or near the upstream end of the steel liner. One or more seepage rings may be required. ASCE (1993) recommends three rings for water pressures above 240 m (800 ft) (see Figure 9-14). A thin liner shell may be provided at the transition, as shown on Figure 9-14 with studs, hooked bars, U-bars, or spirals installed to prevent buckling. Alternatively, ring reinforcement designed for crack control may be provided for a length of about twice the tunnel diameter, reaching at least 900 mm (3 ft) in behind the steel lining. Depending on the character of the rock and the method of construction, a grout curtain may be provided to minimize water flow from the concrete-lined to the steel-lined section through the rock.

g. Bifurcations and other connections. Bifurcations, manifolds, and other connections are generally designed in accordance with the principles of aboveground penstocks, ignoring the presence of concrete surrounding the steel structure. The concrete may be assumed to transfer unbalanced thrust forces to competent rock but is not assumed otherwise to help support internal pressures. Guidance in the design of these structures is found in EM 1110-2-2902, Conduits, Culverts and Pipes, and EM 1110-2-3001. Steel lining connections are usually straight symmetrical or asymmetrical wyes. Right-angle connections should be avoided, as they have higher hydraulic resistance. These connections require reinforcement to replace the tension resistance of the full-circle steel circumference interrupted by the cut in the pipe provided for the connection. The reinforcement can take several forms depending on the pressure in the pipe, the pipe size, and the pipe connection geometry. This is expressed by the pressure-diameter value (PDV), defined as

$$PDV = pd^2/(D \sin^2 \alpha) \tag{9-38}$$

where

p = design pressure, psi

d = branch diameter, in.

D = main diameter, in.

α = branch deflection angle

Depending on the PDV, the reinforcement should be applied as a collar, a wrapper, or a crotch plate. Collars and wrappers are used for smaller pipes where most tunnels would employ crotch plates. These usually take the shape of external plates welded onto the connection between the pipes. The selection of steel reinforcement is made according to Table 9-4. The external steel plate design depends on the geometry and relative pipe sizes. One or more plates may be used, as shown in the examples on Figure 9-15. Because space is limited around the steel lining in a tunnel, it is often practical to replace the steel reinforcement plate with an equivalent concrete reinforcement. For a collar or wrapper, the reinforcement plate should be equal in area to the steel area removed for the connection, except that for PDV between 4,000 lb/in. and 6,000 lb/in., this area should be multiplied by PDV times 0.00025.

Box 9-8. Liner with Stiffeners - Roark's Formula

Liner Thickness t = 1/2, 5/8, 3/4, 7/8, and 1.00 in.
Siffeners: 7/8" x 6" and larger for thicker liners @ 48 in. on centers

Design data:

R_1 = 90 in. - radius to the inside of the liner
t = 1/5, 5/8, 3/4, 7/8, and 1.00 in. - selected range of liner thicknesses
E_s = 30,000,000 psi - modulus of elasticity
υ = 0.3 - Poisson's Ratio
L_1 = 48 in. - spacing of stiffeners
Pcr = "d(t)" - critical (collapsing) pressure for factor of safety F.S. = 1.0

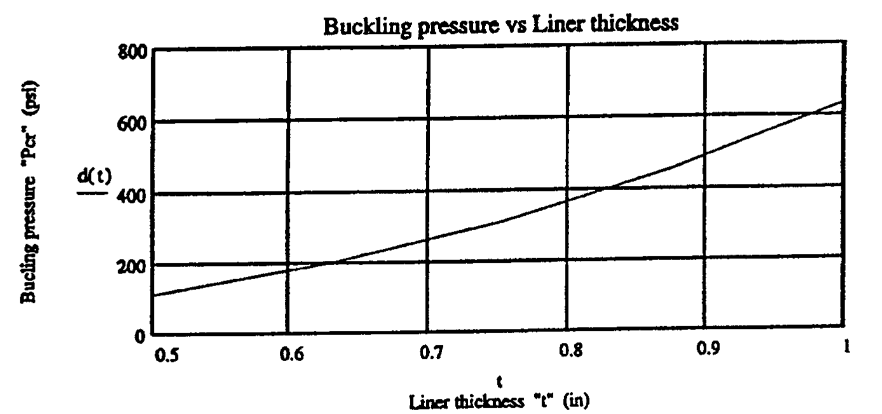
t: = 0.50, 0.625.. 1.00

$R_1 := 90 \quad L_1 := 48 \quad \upsilon_s := 0.3 \quad E_s := 30 \cdot 10^6$

$$d(t) := \frac{0.807 \cdot E_s \cdot t^2}{L_1 \cdot R_1} \cdot \left[\left(\frac{1}{1 - v_s^2}\right)^3 \cdot \frac{t^2}{R_1^2}\right]^{0.25}$$

- critical buckling pressure formula

d(t)
112.081
195.798
308.86
454.076
634.028



External pressures:

t (thickness), in.	Pcr, psi	Pall, psi F.S. = 1.5	Pall, psi F.S. = 2.0
1/2	112	75	66
5/8	196	131	98
3/4	309	206	154
7/8	454	303	227
1.0	634	423	317

Box 9-9. Liner with Stiffeners - R. von Mises's Equation

Liner Thickness t = 0.50 in.
Stiffeners: 7/8" x 6" @ 48 in. on centers

Design data:

r = 90 in. - radius to neutral axis of shell (for practical purposes, radius to outside of shell)
L = 48 in. - length of liner between stiffeners, i.e., center-to-center spacing of stiffeners
t = 0.50 in. - thickness of the liner
E = 30,000,000 psi - modulus of elasticity
υ = 0.3 - Poisson's Ratio
n = number of lobes or waves in the complete circumference at collapse
Pcr = d(n) - critical (collapsing) pressure for factor of safety F.S. = 1.0

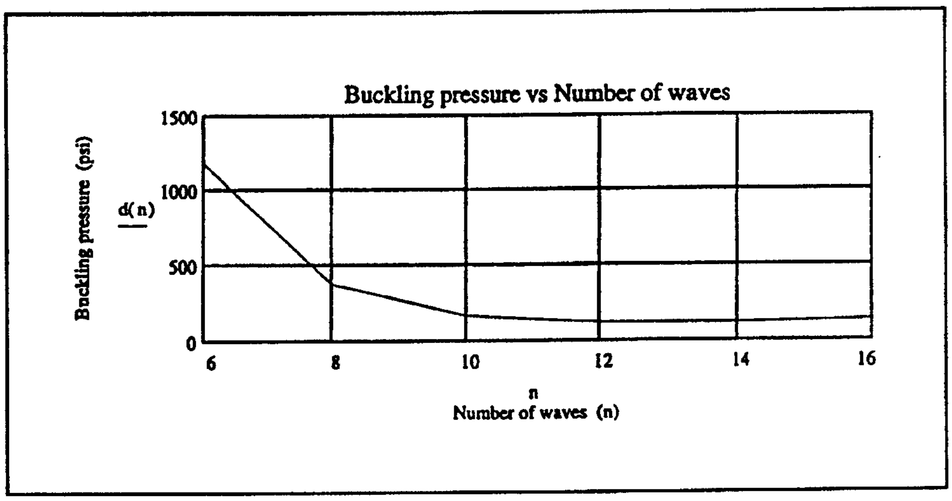
n : = 6, 8 .. 16

t : = 0.50   r : = 90   L : = 48   υ : = 0.3   E : = 30 · 10 ^6^

$$d(n) := \frac{E \cdot \left(\frac{t}{r}\right)}{1 - \nu^2} \cdot \left[\frac{1 - \nu^2}{(n^2 - 1) \cdot \left(\frac{n^2 \cdot L^2}{\pi^2 \cdot r^2} + 1\right)^2}\right] + \frac{E \cdot \left(\frac{t}{r}\right)^3}{12 \cdot (1 - \nu^2)} \cdot \left[n^2 - 1 + \frac{2 \cdot n^2 - 1 - \nu}{\left(\frac{n^2 \cdot L^2}{\pi^2 \cdot L^2}\right) - 1}\right]$$

- critical buckling pressure equation

d(n)
$1.76 \cdot 10^3$
367.522
168.596
121.242
120.951
139.08



External pressures:

Pcr (critical buckling pressure) = 120 psi

Pall (allowable buckling pressure) = 80 psi

Box 9-10. Liner with Stiffeners-Donnell's Equation

Liner Thickness t = 0.50 in.
Stiffeners: 7/8" x 6" @ 48 in. on centers

Design data:

R = 90 in. - shell radius
L = 48 in. - length of liner between stiffeners, i.e., center-to-center spacing of stiffeners
t = 0.50 in. - thickness of the liner
E = 30,000,000 psi - modulus of elasticity
υ = 0.3 - Poisson's Ratio
n = number of lobes or waves in the complete circumference at collapse
Pcr = d(n) - critical (collapsing) pressure for factor of safety F.S. = 1

n := 6, 8 .. 16

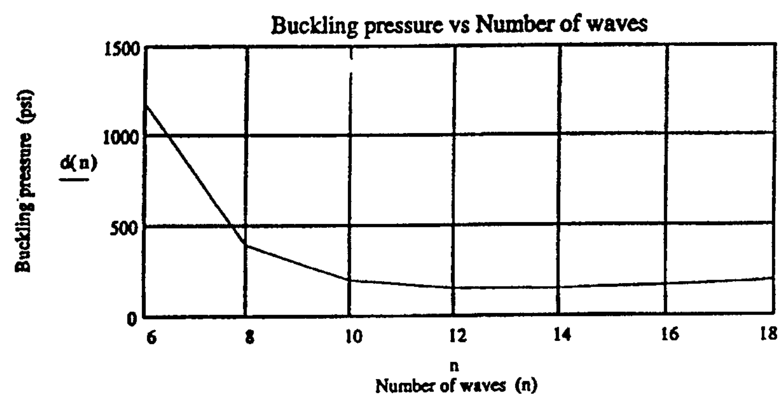
$t := 0.50 \qquad R := 90 \qquad L := 48 \qquad \upsilon := 0.3 \qquad \lambda := \frac{\pi \cdot R}{L} \qquad I_s := \frac{t^3}{12 \cdot (1 - \upsilon^2)}$

$\lambda = 5.89 \qquad I_s = 0.011 \qquad E := 30 \cdot 10^6$

$$d(n) := \frac{E \cdot I_s}{R^3} \cdot \left[\frac{(n^2 + \lambda^2)^2}{n^2}\right] + \frac{E \cdot t}{R} \cdot \left[\frac{\lambda^4}{n^2 \cdot (n^2 + \pi^2)^2}\right]$$

-- critical buckling pressure equation

d(n)
$1.181 \cdot 10^3$
393.553
196.062
148.098
147.148
164.773
191.879



External pressures:

Pcr (critical buckling pressure) = 147 psi

Pall (allowable buckling pressure) = 98 psi (with safety factor F.S. = 1.5)

Box 9-11. Determination of Number of Waves (lobes) at the Liner Collapse

Liner Thicknesses : t = 1/2, 5/8/, 3/4, 7/8 and 1.0 in.
Stiffener spacing @ 48 in. on centers

Design data:

D = 180 in. - tunnel liner diameter
L = 48 in. - spacing of stiffeners
υ = 0.3 - Poisson's Ratio
t = 1/2, 5/8, 3/4, 7/8 and 1.0 in. - selected range of liner thicknesses
n = "d(t)" - number of waves (lobes) in the complete circumference at collapse

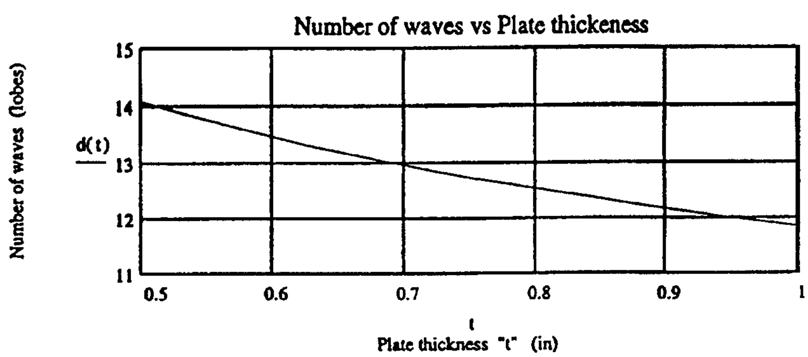
t : = 0.50, 0.625 .. 1.00

D : = 180   L : = 48 υ : = 0.3

$$d(t) := \left[\frac{\dfrac{3\cdot\pi^2}{4\cdot\sqrt{1-\nu^2}}}{\left(\dfrac{L}{D}\right)^2\cdot\left(\dfrac{t}{D}\right)}\right]^{0.25}$$

- - Winderburg and Trilling formula for υ = 0.3

d(t)
14.078
13.314
12.721
12.24
11.838



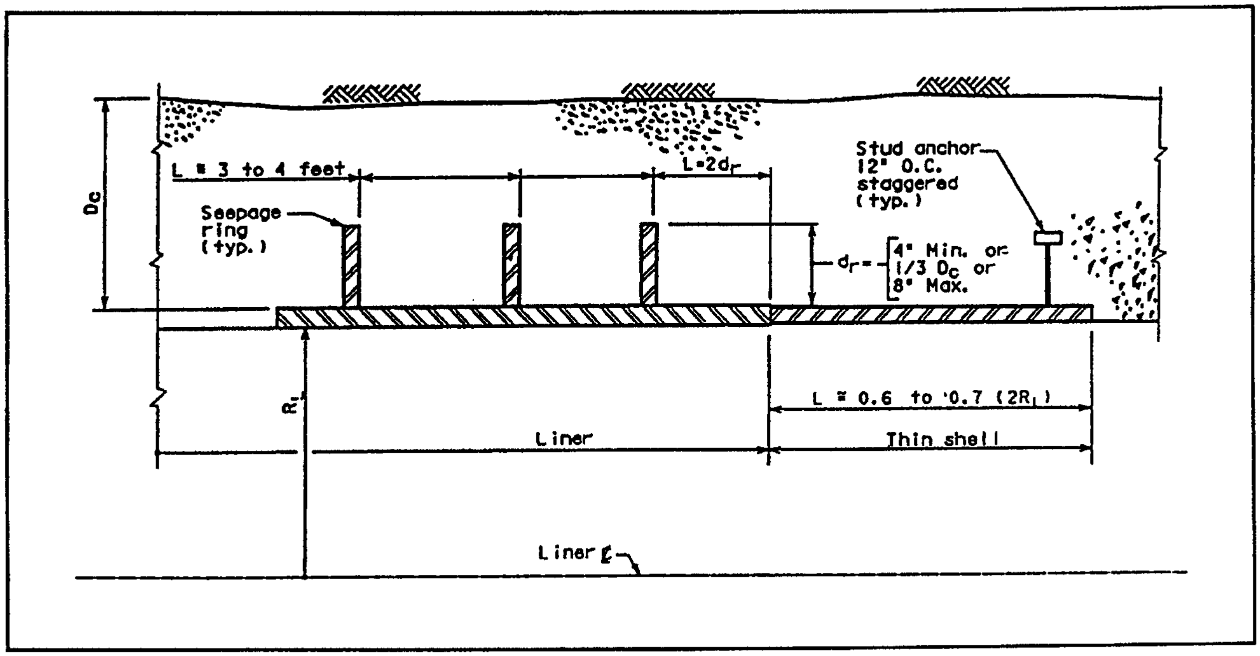


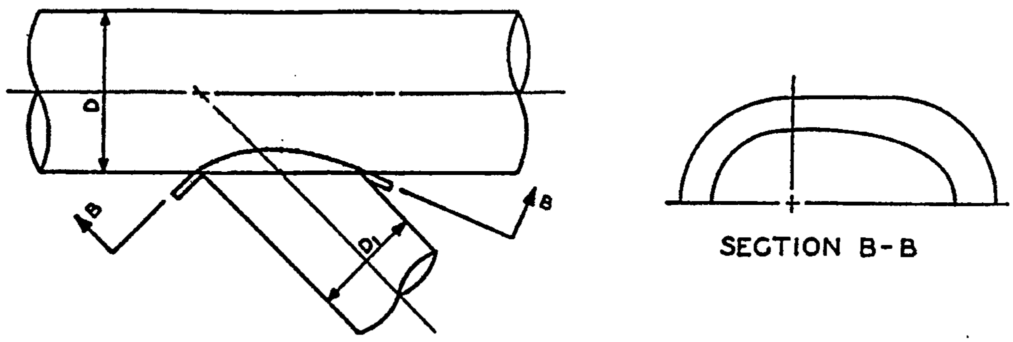
Figure 9-14. Seepage ring and thin shell configuration

Table 9-3
Allowable Buckling Pressures for a 90-in.-diam. Steel Liner With Stiffeners Spaced 48 in.

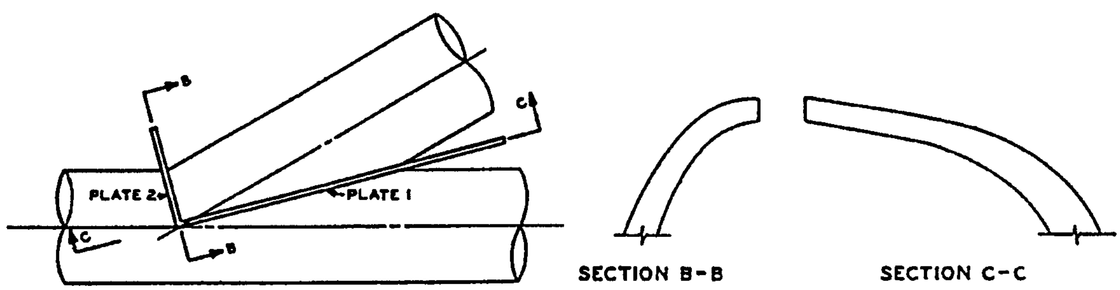
		Plate Thicknesses, in. (ASTM A516-70)				
Analyses/ Formulas	Safety Factor	1/2	5/8	3/4	7/8	1.0
		Allowable Buckling Pressures, psi				
Roark	1.5	75	131	206	303	423
Von Mises	1.5	80	137	218	327	471
Donnell	1.5	98	172	279	424	603
Jacobsen	1.5	84	143	228	348	482

Table 9-4

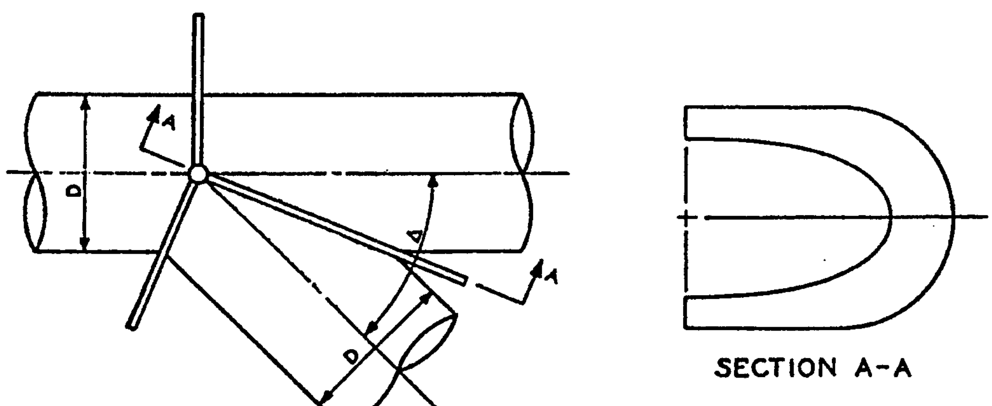
PDV (lb/in.)	>6,000	4,000-6,000	<4,000
d/D	>0.7	Crotch wrapper plate	Wrapper
	<0.7	Crotch collar or plate wrapper	Collar or wrapper



One-Plate Reinforcement



Two-Plate Reinforcement



Three-Plate Reinforcement

Figure 9-15. Steel-lining reinforcement

Chapter 10
Instrumentation and Monitoring

10-1. Purposes of Instrumentation and Monitoring

a. Many construction contracts for underground works in rock incorporate a geotechnical instrumentation and monitoring program as an integral part of the work. To be successful, such monitoring programs must be carried out for well-defined purposes, be well planned, and be supported by competent staff through completion and implementation of results from the monitoring program.

b. Basic principles of instrumentation and monitoring, as well as details of many instruments, can be found in EM 1110-2-4300, Instrumentation for Concrete Structures.

c. Geotechnical monitoring programs are carried out for one or more of the following purposes:

(1) Where initial ground support is selected based on conditions encountered, monitoring can verify the adequacy of the support and indicate if more support is required.

(2) Early monitoring during construction, perhaps in a test area, can help in planning of later construction procedures or help decide whether contingency plans need to be used.

(3) With the NATM (Section 5-5), monitoring of displacements and loads is an essential part of the construction process, providing input to the ongoing process of design and verification during construction.

(4) In the process of determining the adequacy of ground support, monitoring also serves a safety function, warning of the potential for ground failure.

(5) In some cases, decisions regarding final lining installation can be made based on monitoring whether additional reinforcement or a steel lining may be required.

(6) Monitoring may be required to show compliance with environmental requirements (e.g., groundwater lowering, ground settlements, vibrations) or contractual requirements.

(7) Sometimes data can be obtained that are required or useful for the design of other structures (underground powerhouse, dam, other tunnels in the vicinity).

(8) Monitoring can be used to diagnose flaws in the contractor's procedures and indicate better procedures.

(9) Experimental facilities, pilot tunnels, or shafts that are used to obtain data for design of important structures require special types of instrumentation.

(10) On occasion baseline data may be obtained that will be useful in the long-term operation of a facility (e.g., groundwater pressures).

d. The essential ingredients in a successful monitoring program include the following components:

- Definition of need and objective.
- Planning and design.
- Execution of program.
- Interpretation of data.
- Action based on monitoring results.

All of these components must be carefully planned ahead of time. If the data obtained cannot be properly interpreted in a timely fashion, or if no action is foreseen to be taken based on the data, the instrumentation program will have no purpose and should not be implemented. Many tunnels, especially those bored by TBM through reasonably competent rock, require no monitoring program. Large caverns and near-surface structures are more likely to benefit from monitoring programs. Monitoring of safety-related parameters, such as air quality, methane, or radon concentrations, is discussed in Section 5-13. Environmental monitoring is discussed in Section 5-14.

10-2. Planning and Designing the Monitoring Program

Development of a monitoring program begins with defining the purpose(s) of the program and ends with planning how to implement the measurement data. Systematic planning requires a team effort between the designers of the tunnel or shaft and personnel with expertise in the application of

technical instrumentation. Items to consider in planning a successful monitoring program are listed in Table 10-1 and outlined in the following subsections. More comprehensive information is given by Dunnicliff (1988). Specific issues relating to tunnels and underground chamber construction, shafts and portals, and to monitoring in urban environments, are discussed in Section 10-3.

Table 10-1
Items to Consider in Planning a Successful Monitoring Program

1	Define the project conditions
2	Predict mechanisms that control behavior
3	Define the purpose of the instrumentation and monitoring and the questions that need to be answered
4	Select the parameters to be monitored
5	Predict magnitudes of change and set response values for action to be taken
6	Devise remedial actions and arrange for implementation
7	Assign duties and responsibilities for all phases
8	Instrument selection and locations
9	Plan recording of factors that affect measurements
10	Establish procedures to ensure data correctness
11	Prepare instrumentation system design report
12	Plan regular calibration and maintenance
13	Plan data collection and data management

a. Define the project conditions. An engineer or geologist familiar with the project design should be responsible for planning the monitoring program. However, if the program is planned by others, a special effort must be made to become familiar with project conditions including type and layout of the tunnel or shaft, subsurface stratigraphy and engineering properties of subsurface materials, groundwater conditions, status of nearby structures or other facilities, environmental conditions, and planned construction method.

b. Predict mechanisms that control behavior. Before defining a program of instrumentation and monitoring, one or more working hypotheses must be established for mechanisms that are likely to control behavior. Instrumentation should then be planned around these hypotheses. For example, if the purpose is to monitor safety, hypotheses must be established for mechanisms that could lead to rock or support failure.

c. Define the purpose of the instrumentation and monitoring and the questions that need to be answered. Instrumentation should not be used unless there is a valid purpose that can be defended. Peck (1984) states, "The legitimate uses of instrumentation are so many, and the questions that instruments and observation can answer so vital, that we should not risk discrediting their value by using them improperly or unnecessarily." Every instrument should be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation. Before addressing measurement methods themselves, a list should be made of questions that are likely to arise during the construction.

d. Select the parameters to be monitored. Table 10-2 gives a list of parameters that may need to be monitored. It is important to consider which parameters are most significant for each particular situation. For example, if the question is "Is the support overloaded?" stress or load in the support is likely to be the primary parameter of interest. However, recognizing that stress is caused by deformation of the rock, it may also be necessary to monitor deformation. By monitoring both cause and effect, a relationship between the two can often be developed, and action can be taken to remedy any undesirable effect by removing the cause.

Table 10-2
Typical Monitoring Parameters

Project Type	Parameter
Tunnels, underground chambers, shafts and portals	Convergence Crown settlement Floor heave Distribution of deformation behind the rock wall Load in dowels and anchors Stress in concrete or steel linings Groundwater pressure within the rock mass Water pressure acting on lining
Urban environments	Surface settlement Vertical and horizontal deformation of buildings and other structures Vertical and horizontal deformation of the ground at depth Groundwater pressure

e. Predict magnitudes of change, and set response values for action to be taken. Predictions are necessary so that required instrument ranges and required instrument sensitivities or accuracies can be selected. An estimate of the maximum possible value or the maximum value of interest will determine the instrument range, and the minimum value of interest determines the instrument sensitivity or accuracy. Accuracy and reliability are often in conflict since highly accurate instruments may be delicate and/or fragile. A predetermination should be made of

instrumentation readings that indicate the need for remedial action. The concept of green, yellow, and red response values is useful. Green indicates that all is well; yellow indicates the need for cautionary measures including an increase in monitoring frequency; and red indicates the need for timely remedial action.

f. Devise remedial actions and arrange for implementation. Inherent in the use of instrumentation is the necessity to determine, in advance, positive means for solving any problem that may be disclosed by the results of the observations (Peck 1973). If the observations demonstrate that remedial action is needed, that action must be based on appropriate, previously anticipated plans. Personnel involved in the planning process need to devise remedial action plans for site personnel to follow in the event that response values are reached, and design and construction personnel should maintain an open communication channel during construction so that remedial action plans can be discussed between them at any time.

g. Assign duties and responsibilities for all phases. Duties during the monitoring program include planning, instrument procurement, calibration, installation, maintenance, reading, data processing, data presentation, data interpretation, reporting, and deciding on implementation of the results. When duties are assigned for monitoring, the party with the greatest vested interest in the data should be given direct responsibility for producing it accurately.

h. Selection and location.

(1) Reliability is the most desirable feature when selecting monitoring instruments. Lowest first cost of an instrument should not dominate the selection of an instrument. A comparison of the overall cost of procurement, calibration, installation, maintenance, reading, and data processing of the available instruments should be made. The least expensive instrument may not result in least overall cost because it may be less reliable since cost of the instruments themselves is usually a minor part of the overall cost.

(2) Users need to develop an adequate level of understanding of the instruments that they select and often benefit from discussing the application with the manufacturer's staff before selecting instruments. During the discussions, any limitations of the proposed instruments should be determined.

(3) Choosing locations for the instruments should be based on predicted behavior of the tunnel or shaft. The locations should be compatible with the questions and the method of analysis that personnel will use when interpreting the data. A practical approach to selecting instrument locations involves three steps.

(a) First, identify zones of particular concern, such as structurally weak zones or areas that are most heavily loaded, and locate appropriate instrumentation.

(b) Second, select zones (normally cross sections) where predicted behavior is considered representative of behavior as a whole. These zones are regarded as primary instrumented sections. Instruments installed in these zones will provide comprehensive performance data.

(c) Third, because the primary zones may not be truly representative, install simple instrumentation at a number of secondary instrumented sections to serve as indices of comparative behavior. If the behavior at one or more of the secondary sections appears to be significantly different from the primary sections, additional instruments can be installed at the secondary section as construction progresses.

i. Record factors that affect measurements.

(1) For proper interpretation of virtually all site instrumentation data, it is essential to monitor and record all site activities and climatic conditions that can have an effect on the measurements obtained. These include at least the following:

- Progress of excavation (e.g., distance of advancing tunnel face from installation).
- Excavation of adjacent openings, including effects of blasting.
- Installation of lining or other ground support.
- Installation of drains or grouting.
- Unusual events (ground instability, excess water inflows, etc.).
- Continued monitoring of groundwater inflow into the underground space.

(2) Usually, variations in the geology or rock quality have a great effect on monitoring data. While it is generally recommended to map the geology along an important underground facility during construction, it is especially important in the vicinity of extensive monitoring installations.

j. Establish procedures to ensure data correctness. Personnel responsible for monitoring instrumentation must be able to answer the question: "Is the instrument functioning correctly?" They can sometimes determine the answer through visual observations. In critical situations, more than one of the same type of instrument may be used to provide a backup system even when its accuracy is significantly less than that of the primary system. For example, an optical survey can often be used to examine correctness of apparent movement at surface-mounted heads of instruments installed for monitoring subsurface deformation. Repeatability can also give a clue to data correctness. It is often worthwhile to take many readings over a short time span to determine whether a lack of normal repeatability indicates suspect data.

k. Prepare instrumentation system design report. An "Instrument System Design Report" should be written to summarize the planning of all previous steps. This report forces the designer to document all decisions, at which point they can be reviewed to ensure that they meet the needs of the project.

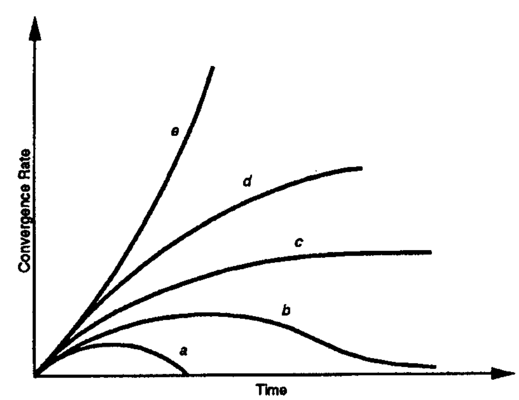
l. Plan regular calibration and maintenance. Regular calibration and maintenance of readout units are required during service life. During the planning process, the instrumentation designers should develop procedures and schedules for regular maintenance of field terminals and accessible embedded components.

m. Plan data collection and data management. Written procedures for collecting, processing, presenting, interpreting, reporting, and implementing data should be prepared before instrumentation work commences in the field. The effort required for these tasks should not be underestimated. Computerized data collection, processing, and presentation procedures have greatly reduced personnel effort, but limitations remain. No computerized system can replace engineering judgment, and engineers must make a special effort to ensure that data are interpreted and reported and that measured effects are correlated with probable causes.

10-3. Monitoring of Tunnel and Underground Chamber Construction

The behavior of a tunnel opening is most drastically manifested in the displacements of the tunnel walls and the rock mass surrounding the tunnel. Convergence of the tunnel walls is by far the most important indicator of tunnel performance and is also relatively easy to measure. Loads, strains, and stresses are generally more difficult to measure, and more difficult to interpret.

a. Displacement and convergence. The absolute value of tunnel convergence can sometimes be predicted, and exceeding this value could be cause for concern; however, the rate of convergence is the more important parameter to watch. Figure 10-1 shows conceptually several time plots of rate of convergence. Curves a and b show decreasing convergence, indicating eventual stability of the structure. If the convergence rate reaches zero, a final lining installation in the tunnel thereafter would receive no load. If the displacement approaches an asymptotic value, the load on the final lining can be reduced by delaying its installation. Very often the time-dependent displacement varies linearly with the log of time, and plots of displacement versus log time can be used to predict long-term performance. Nonuniform convergence is evidence of potential nonuniform loads on a permanent lining. Loads can be inferred from the displacements by back calculation using assumed uniform or nonuniform load distributions so that loads can be compared with those assumed for design, and the adequacy of design can be assessed.



Curves c, d, e with increasing convergence rates generally indicate unacceptable performance.

Figure 10-1. Tunnel convergence rates

(1) The most common convergence measurement is one taken across the horizontal diameter. Vertical measurements are not usually taken due to interference with equipment and traffic. Diametral measurements are also possible. TBM equipment often prevents or seriously hampers attempts at convergence monitoring. In such instances, precision surveying using total stations and reflector targets may be a practical solution. A typical type of response to overstress in a tunnel with a level floor in weak ground such as a clay shale is excessive floor heave.

Monitoring of floor heave is difficult because of traffic and softening due to water flow. Measuring points can be set a distance below the top of the floor and protected, and read with high-precision electronic leveling.

(2) It is often necessary to determine the depth of rock damaged by blasting and the depth of inelastic or creep deformations behind the wall of a tunnel or an underground chamber. Such measurements are especially useful if convergence estimates were part of the basis for ground support or lining design, and if the elastic and inelastic parts of convergence must be differentiated. Data on ground movements behind the tunnel wall are usually obtained using multiposition borehole extensometers (MPBXs). Anchors are attached to the walls of a radially drilled borehole at various distances from the wall. These anchors are connected to a measuring device at the wall that permits a determination of relative displacements between the anchorage points. Comparing such data with theoretical elastic or elastoplastic displacement variations, it is possible to derive parameters for elastic and plastic analysis, and to determine the extent of plastic displacement. This can be important for determining the required depth of dowels or rock bolts. In most tunnels, such measurements would be used only in areas of severe displacements, or for a typical test section, where the data may be applicable to a great length of tunnel. MPBXs are usually more useful in large, complex rock chambers. Where pillars are left between adjacent tunnels, or rock noses left at tunnel wyes, MPBX installations can be used to assess the degree of overstress or the stability within the pillar or nose, as manifested by displacements.

b. Load measurements. In past years, when extensive steel set support was common in tunnels, load cells were often incorporated in selected steel sets. Sometimes load cells were installed between steel sets and the ground to measure loads in and on steel sets to verify design assumptions and add to the database for design of steel sets. These types of measurements are usually not successful, because the presence of the instruments affects the loads measured. A better alternative is to equip steel sets with sets of strain gages for determining strains and loads in the sets. More common are load cells to measure loads on rock anchors in critical tunnel locations. Data from such installations can indicate if anchors need to be supplemented because of excessive loads. Such measurements are beneficially supplemented with MPBX installations to indicate the seat of any ground movements to which high loads may be ascribed. Anchor load cells are primarily installed on tensioned anchors in important large underground chambers.

c. Stress and strain measurements. Strain gages can be embedded in a tunnel lining of shotcrete or cast-in-place concrete to determine stresses and loads within the concrete; however, these installations often fail in their ultimate purpose because strains that occur during curing due to temperature and shrinkage mask the effects of the subsequent stressing of the concrete. Strain gage installations on lattice girders embedded in shotcrete have been more successful. Strain gages have also been used to measure strains in the steel lining of a pressure tunnel. Such measurements can track the performance of the lining in the long term, for example, where nonuniform effects of squeezing or swelling ground or fluctuating groundwater pressures are expected.

d. Measurement of groundwater pressure. There are many instances where groundwater pressures, or the depth of the groundwater table, require monitoring. Piezometers to measure groundwater pressure can be installed in boreholes from the ground surface but can also be installed from within a lined structure or in holes drilled from underground chambers. Examples of situations requiring groundwater monitoring are as follows:

(1) Where groundwater resources must be protected for environmental or economical reasons and the tunnel could act as a drain.

(2) Where groundwater lowering could result in unacceptable formation compaction or consolidation, resulting in ground surface settlements.

(3) Where tunnel leakage could propagate through the rock mass and cause seepage into a powerhouse or slope stability problems in an adjacent valley.

e. Monitoring of shafts and portals.

(1) Portals in rock can suffer instability in the same way as excavated slopes. Loose rock can fall; shallow or deep-seated failures can develop along more-or-less circular slip surfaces or along planes of weakness. Tension cracks can open a distance above the face of the portal slope; if filled with water, such cracks are potentially dangerous because they add to the driving force of a failure mechanism. Portals and slopes, as well as vertical excavated walls, are usually monitored using the following types of installations:

- Settlement points above the slope.

- Survey points or survey reflectors on the face of the slope.
- Inclinometer casings installed vertically from above the slope, probed by inclinometers.
- Horizontal or slightly inclined MPBXs installed from the face of the slope or portal face.
- Monitoring of surface exposures of rock fractures to determine if movement occurs along the fractures.

(2) In general, shafts can be monitored using the same types of devices as tunnels, including convergence measurements and MPBXs. Displacements of large-diameter shaft walls through low-strength overburden are sometimes monitored using inclinometers and other devices, similar to slopes and portals.

f. Monitoring in urban environments. Monitoring of shaft and tunnel construction in urban areas is generally conducted to meet specific environmental requirements and measure environmental effects. In areas of existing structures and utilities, displacements due to tunneling or shaft construction can cause damage. Underground structures in rock do not usually cause undue surface displacement; but on occasions, dewatering occurring during construction can cause consolidation of soft or loose sediments. In such cases, settlement monitoring using surface settlement points, piezometers to measure effects on groundwater pressures, and sometimes inclinometers around shafts may be useful to diagnose unacceptable performance and determine remedial measures. Several other types of monitoring are often required for various environmental purposes, as outlined in Section 5-14. These may include the following general types:

(1) Monitoring of vibrations due to blasting (see Section 5-2) or due to TBM operation.

(2) Monitoring of dust and noise transmitted to habitations in the vicinity due to construction activities and related construction traffic.

(3) Monitoring the chemical quality and silt content of the effluent water from the construction site; discovery of pollutants encountered in excavated materials or pumped water.

(4) Monitoring of air quality in general.

g. Data collection and interpretation. Personnel should take the first step in determining whether the instrument data are accurate and the instrument is functioning correctly by comparing the latest readings with previous readings. From this comparison, the personnel can identify any significant changes. If response values have been reached, the plan for remedial action should be implemented. During data collection, all factors that may influence measured data should be recorded and damage, deterioration, or malfunction of instruments noted. The first aim of data processing and presentation is to provide a rapid assessment of data to detect changes that require immediate action. Data collection personnel are usually responsible for this task. The second aim is to summarize and present the data to show trends and compare observed with predicted behavior to determine the appropriate action. After data have been processed, plots of data are prepared with plots of predicted behavior and causal data often included on the same axes.

h. Interpretion of data. The method of data interpretation is guided by the original purpose for the monitoring program. Communication channels between design and field personnel should remain open. Design engineers who framed the questions that need to be answered should continue to interact with the field engineers who provide the data. The data should be evaluated to determine reading correctness and to detect changes requiring immediate action. Data readings must be correlated with other factors to determine cause and effect relationships and to study the deviation of the readings from the predicted behavior. When faced with data that, on first sight, do not appear to be reasonable, there is a temptation to reject the data as false. However, such data may be real and carry an important message. A significant question to ask is: "Can I think of a hypothesis that is consistent with the data?" The resultant discussion, together with the procedures used for ensuring data correctness, will often lead to an assessment of data validity.

Appendix A
References

A-1. Required Publications

Note: References used in this manual are available on interlibrary loan from the Research Library, ATTN: CEWES-IM-MI-R, U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

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Geophysical Exploration

EM 1110-1-1804
Geotechnical Investigations

EM 1110-1-2907
Rock Reinforcement

EM 1110-2-2000
Standard Practice for Concrete

EM 1110-2-2005
Standard Practice for Shotcrete

EM 1110-2-2902
Conduits, Culverts and Pipes

EM 1110-2-3506
Grouting Technology

A-2. Related Publications

ER 415-1-11
Biddability, Constructibility and Operability

ER 1110-1-1801
Construction Foundation Report

ER 1110-1-8100
Laboratory Investigations and Testing

ER 1110-2-100
Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures

ER 1110-2-1150
Engineering and Design for Civil Works Projects

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ER 1110-2-1806
Earthquake Design and Evaluation of Civil Works Projects

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EM 1110-1-1003
NAVSTAR Global Positioning System Surveying

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EM 1110-2-2104
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Appendix B
Frequently Used Tunneling Terms

ANFO - Ammonium nitrate mixed with fuel oil used as an explosive in rock excavation.

Active reinforcement - Reinforcing element that is prestressed or artificially tensioned in the rock mass when installed.

Alluvium - A general term for recent deposits resulting from streams.

Aquiclude - **1.** Rock formation that, although porous and capable of absorbing water slowly, does not transmit water fast enough to furnish an appreciable supply for a well or spring. **2.** An impermeable rock formation that may contain water but is incapable of transmitting significant water quantities. Usually functions as an upper or lower boundary of an aquifer.

Aquifer - **1.** A water-bearing layer of permeable rock or soil. **2.** A formation, a group of formations, or a part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells and springs.

Aquitard - A formation that retards but does not prevent water moving to or from an adjacent aquifer. It does not yield water readily to wells or springs, but may store groundwater.

Artesian condition - Groundwater confined under hydrostatic pressure. The water level in an artesian well stands above the top of the artesian water body it taps. If the water level in an artesian well stands above the land surface, the well is a flowing artesian well.

Average lithostatic gradient - An approximation of the increase in lithostatic stress with depth.

Back - The surface of the tunnel excavation above the spring line; also, roof (see, also, crown)

Backfill - Any material used to fill the empty space between a lining system and excavated rock or soil surface.

Bench - A berm or block of rock within the final outline of a tunnel that is left after a top heading has been excavated.

Bit - A star or chisel-pointed tip forged or screwed (detachable) to the end of a drill steel.

Blocking - Wood or metal blocks placed between the excavated surface of a tunnel and the bracing system, e.g., steel sets. Continuous blocking can also be provided by shotcrete.

Bootleg or Socket - That portion or remainder of a shothole found in a face after a blast has been fired.

Brattice (brattishing) - A partition formed of planks or cloth in a shaft or gallery for controlling ventilation.

Breast boarding - Partial or complete braced supports across the tunnel face that hold soft ground during tunnel driving.

Bulkhead - A partition built in an underground structure or structural lining to prevent the passage of air, water, or mud.

Burn cut - Cut holes for tunnel blasting that are heavily charged, close together, and parallel. About four cut holes are used that produce a central, cylindrical hole of completely shattered rock. The central or burn cut provides a free face for breaking rock with succeeding blasts.

Cage - A box or enclosed platform used for raising or lowering men or materials in a shaft.

Calcareous - Containing calcium carbonate.

Calcite - A mineral predominantly composed of calcium carbonate, with Moh's hardness 3.

California switch - A portable combination of siding and switches superimposed on the main rail track in a tunnel.

Center core method - A sequence of excavating a tunnel in which the perimeter above the invert is excavated first to permit installation of the initial ground support. One or a series of side and crown drifts may be utilized. The center core is excavated after the initial ground support is installed.

Chemical grout - A combination of chemicals that gel into a semisolid after they are injected into the ground to solidify water-bearing soil and rocks.

Cherry picker - A gantry crane used in large tunnels to pick up muck cars and shift a filled car from a position next to the working face over other cars to the rear of the train.

Cohesion - A measure of the shear strength of a material along a surface with no perpendicular stress applied to that surface.

Conglomerate - A sedimentary rock mass made up of rounded to subangular coarse fragments in a matrix of finer grained material.

Controlled blasting - Use of patterned drilling and optimum amounts of explosives and detonating devices to control blasting damage.

Cover - Perpendicular distance to nearest ground surface from the tunnel.

Crown - The highest part of a tunnel.

Cut-and-cover - A sequence of construction in which a trench is excavated, the tunnel or conduit section is constructed, then covered with backfill.

Cutterhead - The front end of a mechanical excavator, usually a wheel on a tunnel boring machine, that cuts through rock or soft ground.

Delays - Detonators that explode at a suitable fraction of a second after passage of the firing current from the exploder. Delays are used to ensure that each charge will fire into a cavity created by earlier shots in the round.

Disk cutter - A disc-shaped cutter mounted on a cutterhead.

Drag bit - A spade-shaped cutter mounted on a cutterhead.

Drift - An approximately horizontal passageway or portion of a tunnel. In the latter sense, depending on its location in the final tunnel cross section, it may be classified as a "crown drift," "side drift," "bottom drift," etc. A small tunnel driven ahead of the main tunnel.

Drifter - A rock drill mounted on column, bar, or tripod, used for drilling blast holes in a tunnel face, patented by J. G. Leyner, 1897.

Drill-and-blast - A method of mining in which small-diameter holes are drilled into the rock and then loaded with explosives. The blast from the explosives fragments and breaks the rock from the face so that the rock can be removed. The underground opening is advanced by repeated drilling and blasting.

Drill steel - See steel, drill.

Elastic - Describes a material or a state of material where strain or deformation is recoverable, nominally instantaneously but actually within certain tolerances and within some arbitrary time. Capable of sustaining stress without permanent deformation.

Elastic rock zone - The zone outside the relaxed rock zone where excavation has altered the in situ stress field. Rock in the elastic zone undergoes recoverable elastic deformation.

Erector arm - Swing arm on tunnel boring machine or shield, used for picking up supports and setting them in position.

Extrados - The exterior curved surface of an arch.

Face - The advance end or wall of a tunnel, drift, or other excavation at which work is progressing.

Final ground or rock support - Support placed to provide permanent stability, usually consisting of rock reinforcement, shotcrete, or concrete lining. May also be required to improve fluid flow, ensure water tightness, or improve appearance of tunnel surface.

Finite element method - The representation of a structure as a finite number of two-dimensional and/or three-dimensional components called finite elements.

Firm ground - Stiff sediments or soft sedimentary rock in which the tunnel heading can be advanced without any, or with only minimal, roof support; the permanent lining can be constructed before the ground begins to move or ravel.

Forepole - A pointed board or steel rod driven ahead of timber or steel sets for temporary excavation support.

Forepoling - Driving forepoles ahead of the excavation, usually supported on the last steel set or lattice girder erected, and in an array that furnishes temporary overhead protection while installing the next set.

Full-faceheading - Excavation of the whole tunnel face in one operation.

Gouge zone - A layer of fine, wet, clayey material occurring near, in, or at either side of a fault or fault zone.

Grade - Vertical alignment of the underground opening or slope of the vertical alignment.

Granite - A coarse-grained, plutonic (intrusive) igneous rock with a general composition of quartz (10-30 percent), feldspar (50-80 percent), mostly potassium feldspar, and mafic minerals such as biotite (10-20 percent).

Granodiorite - A coarse-grained crystalline, intrusive rock with a general composition of quartz (10-20 percent), feldspar (50-60 percent), mostly sodium-rich feldspar, and mafic minerals such as biotite (20-30 percent).

Ground control - Any technique used to stabilize a disturbed or unstable rock mass.

Ground stabilization - Combined application of ground reinforcement and ground support to prevent failure of the rock mass.

Ground support - Installation of any type of engineering structure around or inside the excavation, such as steel sets, wooden cribs, timbers, concrete blocks, or lining, which will increase its stability. This type of support is external to the rock mass.

Grout - Neat cement slurry or a mix of equal volumes of cement and sand that is poured into joints in masonry or injected into rocks. Also used to designate the process of injecting joint-filling material into rocks. See grouting.

Grouting - 1. Injection of fluid grout through drilled holes, under pressure, to fill seams, fractures, or joints and thus seal off water inflows or consolidate fractured rock ("formation grouting"). 2. Injection of fluid grout into annular space or other voids between tunnel lining and rock mass to achieve contact between the lining and the surrounding rock mass ("skin" or "contact" grouting). 3. Injection of grout in tail/void behind prefabricated, segmental lining ("backfill grouting"). 4. The injection under relatively high pressures of a very stiff, "zero-slump" mortar or chemical grout to displace and compact soils in place ("compaction grouting").

Gunite - See shotcrete.

Heading - The wall of unexcavated rock at the advance end of a tunnel. Also used to designate any small tunnel and a small tunnel driven as a part of a larger tunnel.

Heading and bench - A method of tunneling in which a top heading is excavated first, followed by excavation of the horizontal bench.

Ho-ram - A hydraulically operated hammer, typically attached to an articulating boom, used to break hard rock or concrete.

Hydraulic jacking - Phenomenon that develops when hydraulic pressure within a jacking surface, such as a joint or bedding plane, exceeds the total normal stress acting across the jacking surface. This results in an increase of the aperture of the jacking surface and consequent increased leakage rates, and spreading of the hydraulic pressures. Sometimes referred to as hydraulic fracturing.

Indurated - Said of compact rock or soil, hardened by the action of pressure, cementation, and heat.

Initial ground or rock support - Support required to provide stability of the tunnel opening, installed directly behind the face as the tunnel or shaft excavation progresses, and usually consisting of steel rib or lattice girder sets, shotcrete, rock reinforcement, or a combination of these.

Intrados - The interior curved surface of an arch.

Invert - On a circular tunnel, the invert is approximately the bottom 90 deg of the arc of the tunnel; on a square-bottom tunnel, it is the bottom of the tunnel.

Invert strut - The member of a set that is located in the invert.

Joint - A fracture in a rock along which no discernable movement has occurred.

Jumbo - A movable machine containing working platforms and drills, used for drilling and loading blast holes, scaling the face, or performing other work related to excavation.

Jump set - Steel set or timber support installed between overstressed sets.

Lagging - Wood planking, steel channels, or other structural materials spanning the area between sets.

Lifters - Shot holes drilled near the floor of a tunnel and fired after the burn or wedge cut holes and relief holes.

Line - Horizontal or planar alignment of the underground opening.

Liner Plates - Pressed steel plates installed between the webs of the ribs to make a tight lagging, or bolted together outside the ribs to make a continuous skin.

Lithology - The character of a rock described in terms of its structure, color, mineral composition, grain size, and arrangement of its component parts.

Lithostatic pressure - The vertical pressure at a point in the earth's crust that is equal to the pressure that would be exerted by a column of the overlying rock or soil.

Mine straps - Steel bands on the order of 12 in. wide and several feet long designed to span between rock bolts and provide additional rock mass support.

Mining - The process of digging below the surface of the ground to extract ore or to produce a passageway such as a tunnel.

Mixed face - The situation when the tunnel passes through two (or more) materials of markedly different characteristics and both are exposed simultaneously at the face (e.g., rock and soil, or clay and sand).

Moh's hardness scale - A scale of mineral hardness, ranging from 1 (softest) to 10 (hardest).

Muck - Broken rock or earth excavated from a tunnel or shaft.

Open cut - Any excavation made from the ground surface downward.

Overbreak - The quantity of rock that is actually excavated beyond the perimeter established as the desired tunnel outline.

Overburden - The mantle of earth overlying a designated unit; in this report, refers to soil load overlying the tunnel.

Passive reinforcement - Reinforcing element that is not prestressed or tensioned artificially in the rock, when installed. It is sometimes called rock dowel.

Pattern reinforcement or pattern bolting - The installation of reinforcement elements in a regular pattern over the excavation surface.

Penstock - A pressure pipe that conducts water to a power plant.

Phreatic surface - That surface of a body of unconfined ground water at which the pressure is equal to that of the atmosphere.

Pillar - A column or area of coal or ore left to support the overlaying strata or hanging wall in mines.

Pilot drift or tunnel - A drift or tunnel driven to a small part of the dimensions of a large drift or tunnel. It is used to investigate the rock conditions in advance of the main tunnel excavation, or to permit installation of ground support before the principal mass of rock is removed.

Piping - The transport of silt or sand by a stream or water through (as an embankment), around (as a tunnel), or under (as a dam) a structure.

Plastic - Said of a body in which strain produces continuous, permanent deformation without rupture.

Pneumatically applied mortar or concrete - See shotcrete.

Portal - The entrance from the ground surface to a tunnel.

Powder - Any dry explosive.

Prereinforcement - Installation of reinforcement in a rockmass before excavation commences.

Prestressed rock anchor or tendon - Tensioned reinforcing elements, generally of higher capacity than a rock bolt, consisting of a high-strength steel tendon (made up of one or more wires, strands, or bars) fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock at the other end.

Principal stress - A stress that is perpendicular to one of three mutually perpendicular planes that intersect at a point on which the shear stress is zero; a stress that is normal to a principal plane of stress. The three principal stresses are identified as least or minimum, intermediate, and greatest or maximum.

Pull - The advance during the firing of each complete round of shot holes in a tunnel.

P-waves - Compressional waves.

Pyramid cut - A method of blasting in tunneling or shaft sinking in which the holes of the central ring (cut holes) outline a pyramid, their toes being closer together than their collars.

Quartz - A mineral composed of silicon and oxygen, with Moh's hardness of 7.

Raise - A shaft excavated upwards (vertical or sloping). It is usually cheaper to raise a shaft than to sink it since the cost of mucking is negligible when the slope of the raise exceeds 40° from the horizontal.

Ravelling Ground - Poorly consolidated or cemented materials that can stand up for several minutes to several hours at a fresh cut, but then start to slough, slake, or scale off.

Recessed rock anchor - A rock anchor placed to reinforce the rock behind the final excavation line after a portion of the tunnel cross section is excavated but prior to excavating to the final line.

Relievers or relief holes - The holes fired after the cut holes and before the lifter holes or rib (crown, perimeter) holes.

Rib - **1.** An arched individual frame, usually of steel, used in tunnels to support the excavation. Also used to designate the side of a tunnel. **2.** An H- or I-beam steel support for a tunnel excavation (see Set).

Rib holes - Holes drilled at the side of the tunnel of shaft and fired last or next to last, i.e., before or after lifter holes.

Roadheader - A mechanical excavator consisting of a rotating cutterhead mounted on a boom; boom may be mounted on wheels or tracks or in a tunnel boring machine.

Rock bolt - A tensioned reinforcement element consisting of a rod, a mechanical or grouted anchorage, and a plate and nut for tensioning by torquing the nut or for retaining tension applied by direct pull.

Rock dowel - An untensioned reinforcement element consisting of a rod embedded in a grout-filled hole.

Rock mass - In situ rock, composed of various pieces the dimensions of which are limited by discontinuities.

Rock reinforcement - The placement of rock bolts, rock anchors, or tendons at a fairly uniform spacing to consolidate the rock and reinforce the rock's natural tendency to support itself. Also used in conjunction with shotcrete on the rock surface.

Rock reinforcement element - A general term for rock bolts, tendons, and rock anchors.

Rock support - The placement of supports such as wood sets, steel sets, or reinforced concrete linings to provide resistance to inward movement of rock toward the excavation.

Round - A group of holes fired at nearly the same time. The term is also used to denote a cycle of excavation consisting of drilling blast holes, loading, firing, and then mucking.

Scaling - The removal of loose rock adhering to the solid face after a shot has been fired. A long scaling bar is used for this purpose.

Segments - Sections that make up a ring of support or lining; commonly steel or precast concrete.

Set - The complete frame of temporary support, usually of steel or timber, inserted at intervals in a tunnel to support the ground as a heading is excavated (see Rib).

Shaft - An elongated linear excavation, usually vertical, but may be excavated at angles greater than 30 deg from the horizontal.

Shear - A deformation that forms from stresses that displace one part of the rock past the adjacent part along a fracture surface.

Shield - A steel tube shaped to fit the excavation line of a tunnel (usually cylindrical) and used to provide support for the tunnel; provides space within its tail for erecting supports; protects the men excavating and erecting supports; and if breastboards are required, provides supports for them. The outer surface of the shield is called the shield skin.

Shield tail (or skirt) - An extension to the rear of the shield skin that supports soft ground and enables the tunnel primary lining to be erected within its protection.

Shotcrete - Concrete pneumatically projected at high velocity onto a surface; pneumatic method of applying a lining of concrete; this lining provides tunnel support and can serve as the permanent lining.

Shove - The act of advancing a TBM or shield with hydraulic jacks.

Skip - A metal box for carrying rock, moved vertically or along an incline.

Spall - A chip or splinter of rock. Also, to break rock into smaller pieces.

Spiles - Pointed boards or steel rods driven ahead of the excavation, (similar to forepoles).

Spoil - See muck.

Spot reinforcement or spot bolting - The installation of reinforcement elements in localized areas of rock instability or weakness as determined during excavation. Spot reinforcement may be in addition to pattern reinforcement or internal support systems.

Spring line - The point where the curved portion of the roof meets the top of the wall. In a circular tunnel, the spring lines are at opposite ends of the horizontal center line.

Squeezing ground - Material that exerts heavy pressure on the circumference of the tunnel after excavation has passed through that area.

Stand-up-time - The time that elapses between the exposure of rock or soil in a tunnel excavation and the beginning of noticeable movements of the ground.

Starter tunnel - A relatively short tunnel excavated at a portal in which a tunnel boring machine is assembled and mobilized.

Steel, drill - A chisel or star-pointed steel rod used in making a hole in rock for blasting. A steel rod used to transmit thrust or torque from a power source, compressed air or hydraulic, to the drill bit.

Stemming - Material used for filling a blasting hole to confine the charge or explosive. Damp sand, damp sand mixed with clay, or gypsum plaster are examples of materials used for this purpose.

Struts - Compression supports placed between tunnel sets.

TBM - Tunnel boring machine.

Tail void - The annular space between the outside diameter of the shield and the outside of the segmental lining.

Tie rods - Tension members between sets to maintain spacing. These pull the sets against the struts.

Tight - Rock remaining within the minimum excavation lines after completion of a round—that is, material that would make a template fit tight. "Shooting tights" requires closely placed and lightly loaded holes.

Timber sets - The complete frames of temporary timbering inserted at intervals to support the ground as heading is excavated.

Top heading - 1. The upper section of the tunnel. 2. A tunnel excavation method where the complete top half of the tunnel is excavated before the bottom section is started.

Tunnel - An elongated, narrow, essentially linear excavated underground opening with a length greatly exceeding its width or height. Usually horizontal but may be driven at angles up to 30 deg.

Tunnel Boring Machine (TBM) - A machine that excavates a tunnel by drilling out the heading to full size in one operation; sometimes called a mole. The tunnel boring machine is typically propelled forward by jacking off the excavation supports emplaced behind it or by gripping the side of the excavation.

Voussoir - A section of an arch. One of the wedge-shaped pieces of which an arch is composed or assumed to be composed for purposes of analysis.

Walker - One who supervises the work of several gangs.

Water table - The upper limit of the ground saturated with water.

Weathering - Destructive processes, such as the discoloration, softening, crumbling, or pitting of rock surfaces brought about by exposure to the atmosphere and its agents.

Appendix C
Tunnel Boring Machine Performance Concepts and Performance Prediction

This appendix provides information for tunnel designers concerning TBM performance specifications, test data for performance estimates, and estimating costs for TBM tunnels.

C-1. TBM Design and Performance Concepts

The focus of a site investigation and testing program is not just to support the tunnel design. Testing results and recommendations made must also sensitize the contractor to the site conditions before construction, a perspective that permits estimation of cost and schedule and supports the selection of appropriate excavation equipment. The tests used to characterize rock for excavation purposes are often different from tests utilized in other civil works and may depend on the excavation method. For comparison of several alignments, a simple inexpensive test may be sensitive enough to detect differences in boreability, identify problem areas, and give an estimate of thrust and torque requirements.

a. Principles of disc cutting. TBM design and performance predictions require an appreciation of basic principles of disc cutting. Figure C-1 illustrates the action of disc cutting tools involving inelastic crushing of rock material beneath the cutter disc and chip breakout by fracture propagation to an adjacent groove. The muck created in this process includes fine materials from crushing and chips from fracture. The fines are active participants in disc wear. Rock chips have typical dimensions of 15- to 25-mm thickness, widths on the order of the cutter disc groove spacing, and lengths on the order of two to four times the chip width. For efficient disc cutting by a TBM, several items are important including the following:

- The cutter indenting, normal force, and penetration must be sufficient to produce adequate penetration for kerf interaction and chip formation.

- Adjacent grooves must be close enough so that lateral cracks can interact and extend to create a chip.

- There must be a disc force component adequate to maintain cutter movement, in spite of the rolling resistance or drag associated with the penetration process.

b. Normal forces. Disc penetration is affected by the applied TBM thrust. The average thrust, or normal force (F_n), per cutter is calculated as:

$$F_n = N_c \, p_c' \, \pi \, d_c^2 \, / \, (4 \, n) \qquad \text{(C-1)}$$

where N_c is the number of thrust cylinders; p_c' is the net applied hydraulic pressure; d_c is the diameter of each cylinder piston; and n is the number of cutters in the array. Thrust delivered to the cutters is less than that calculated based on operating hydraulic pressure. If the backup system for a TBM is towed behind the TBM during mining, then this loss of thrust should be subtracted, as should friction losses from contact between the machine and the rock. For full shields this loss can be very high and may ultimately stop forward progress if ground pressures on the shield are larger than can be overcome by available thrust. The net average cutter normal force can easily be 40 percent less than the calculated gross force. For very hard rock, thrust limits may severely restrict the penetration rate.

c. Disc rolling force. Disc rolling is affected by supplied machine power and cutterhead rotation. The average rolling force per cutter, F_r, is calculated as:

$$F_r = P' \, / \, (2\pi \; n \; r \; R_c) \qquad \text{(C-2)}$$

where P' is the net delivered power; r is the cutterhead rotation rate (rpm); and R_c is the weighted average cutter distance from the center of rotation. Losses on installed power can also be significant, and overall torque system efficiency is generally about 75 percent. Available F_r can be further reduced when motor problems temporarily decrease the available torque, sticky muck clogs the cutterhead and muck buckets resulting in torque losses from friction and drag against rotation, or with a "frozen" or blocked cutter with a seized bearing. In fact, for many TBMs operated in weak to moderately strong rock, the torque capacity limits the penetration rate. This influence is decreased in recent TBMs designed with variable cutterhead rotation rates and higher powered motors. Load capacity of a sidewall gripper system can also limit the level of thrust and torque that can be applied. With weak rock, the grippers may slide or develop local bearing capacity failure in the sidewall rock. In weak rock, wood cribbing may be required if overbreak is more extensive than the gripper cylinder stroke. These problems are particularly severe when mining from weak into hard rock when high thrust is desired for efficient cutting the grippers must bear on low-strength rock. For shielded TBMs, the strength of the lining may limit operating thrust and torque.

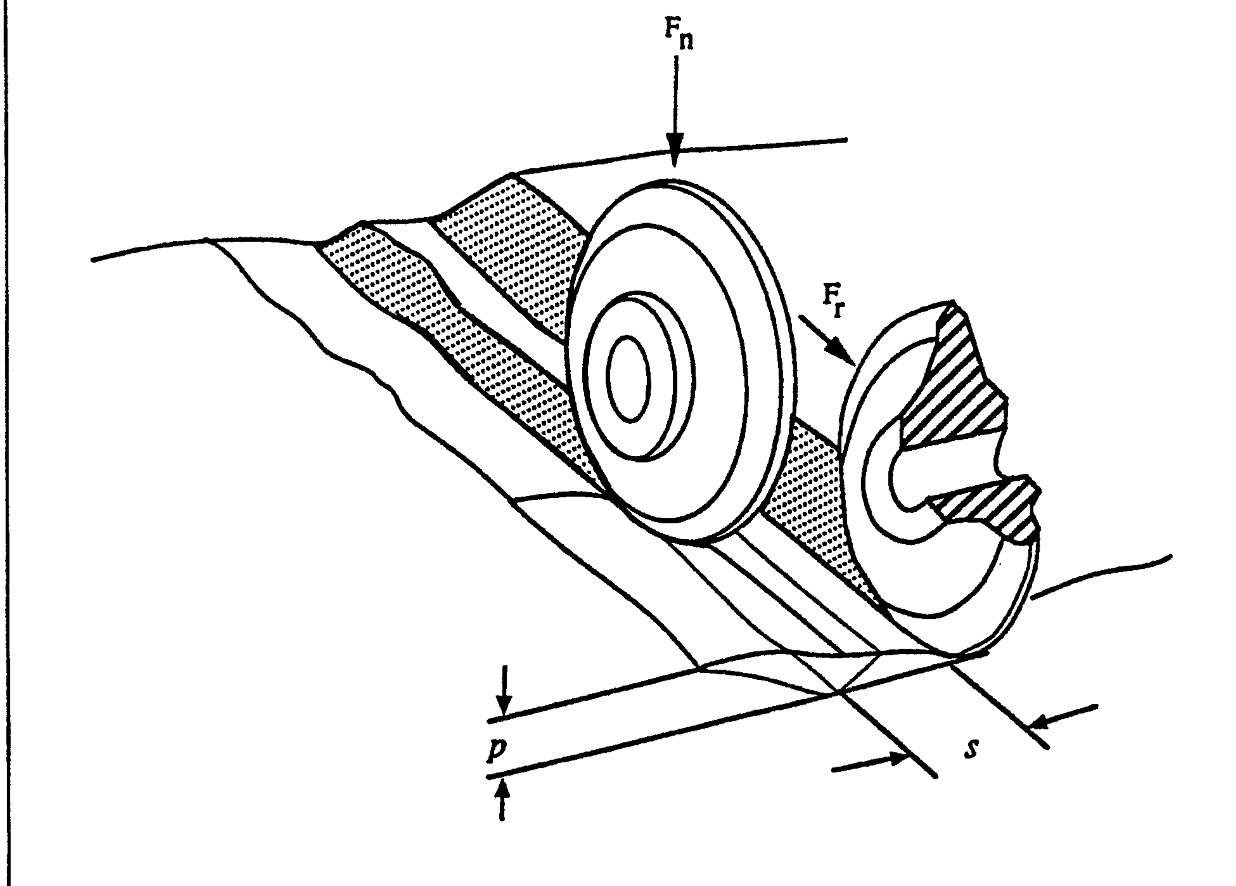

Figure C-1. Disc force and geometry for Kerf cutting

d. Disc force penetration index. TBM operating conditions are not uniform, and it is not likely that the disc forces calculated above are actually developed for any particular cutter. However, it is convenient to develop a model for disc force prediction in the context of these average forces, as well as average disc spacing *(s)* and penetration per revolution (*PRev*). The interaction of F_n and F_r, and the resulting penetration is indicated in Figure C-2. The changing slope corresponds to a transition in dominance between crushing and chip formation and has been called the "critical thrust": unless force of this magnitude can be applied, chipping between grooves will not occur. The critical thrust is directly related to rock strength or hardness and increases with cutter spacing and disc edge width. Although these force/penetration relationships are known to be nonlinear, several parameters have been defined based on ratios derived from force/penetration plots. The ratio of F_r to F_n has been defined as the cutting coefficient (C_c), and the ratio of F_n to *PRev* is defined as the penetration index (R_f). Therefore:

$$C_c = \frac{F_r}{F_n} \text{ and } R_f = \frac{F_n}{PRev} \qquad \text{(C-3)}$$

e. Research on TBM cutting mechanics has yielded the following important observations:

- *PRev* is primarily controlled by F_n; i.e., with sufficient delivered power, cutterhead rpm does not strongly affect PRev.

- Optimized cutting is possible when the ratio of spacing(s) to *PRev* (*s/p*) is on the order of about 8 to 20 for a wide variety of rock units.

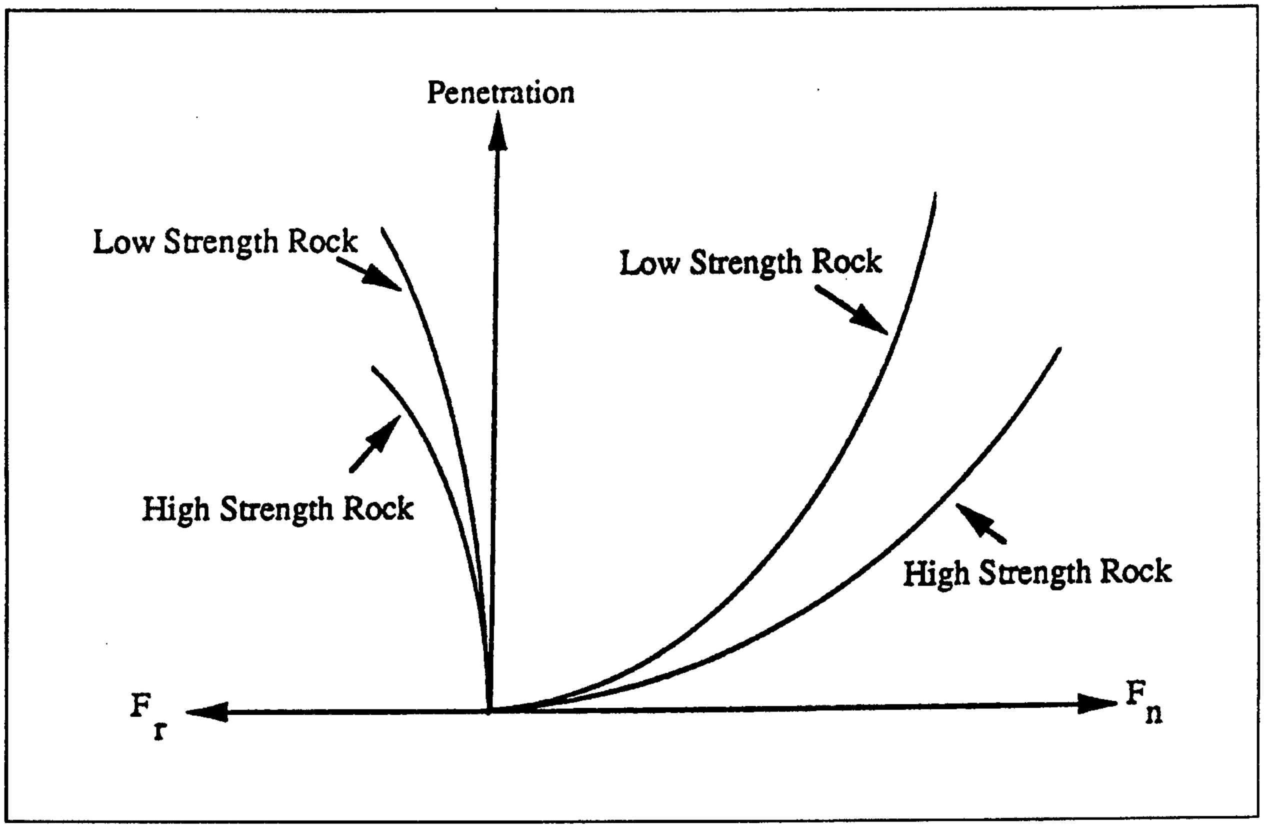

Figure C-2. General plot of disc cutter force variation with penetration for high- and low-strength rocks

- A less than optimum but still satisfactory cutting rate *s/p* ratio may occur in weaker rock due to high penetrations at lower cutter forces.

- For strong rock, high critical thrust results in reduced penetration and increased *s/p* ratios, and acceptable mining rates are difficult to achieve.

- For porous or microfractured rock, indentation results in large volumes of crushed and potentially abrasive material and reduced chip formation.

C-2. TBM Penetration Rate Prediction From Intact Rock Properties

The most important independent variables for TBM design include installed power, cutterhead rpm, thrust, and disc spacing. Each parameter influences the resulting penetration rate. In practice, average disc spacing has been designed in a limited range between 60 and 90 mm. Fixed design conditions include disc rolling velocity and disc tool loading limits. Given accepted limits on disc velocity and loading and the general range of target *s/p* ratios used in practice, a method to predict relationships between F_n, F_r, and *PRev* would permit a TBM design with adequate power and thrust to achieve desired penetration rates.

a. Prediction methods. Many efforts have been made to correlate laboratory index test results to TBM penetration rate. Prediction equations are either empirically derived or developed with a theoretical basis using force equilibrium or energy balance theories. Simplified assumptions of disc indentation geometry and contact zone stress distribution are made, and coefficients derived from correlations with case history information are used. Most prediction methods agree on trends, but empirical methods are case-specific in terms of geology and machine characteristics. However, a general statement of caution about the case history databases should be made. Prediction methods that do not consider operating conditions of thrust and torque cannot be applied to projects where equipment operations vary. The condition of the cutters can also have a significant effect on performance, since worn or blunted discs present wider contact areas on indentation and require higher forces for a given level of penetration. Some data bases include performance with single, double, and triple

disc cutters, a variation that greatly affects disc edge loading and spacing/penetration ratios. Finally, low-thrust and low-torque mining through poor ground or alignment curves may result in reduced penetration rates.

b. Penetration index tests. As examples of index tests used in correlations, several prediction approaches utilize static indentation tests performed on confined rock specimens. A second group of index tests can be called "hardness" tests, including Shore hardness, Scleroscope hardness, Taber abrasion hardness, Schmidt hammer rebound hardness (H_R), and Total Hardness (H_T), which is calculated as the product of H_R and the square root of the Taber abrasion hardness. Dynamic impact tests have also been developed for application to TBM performance prediction. These include Rock Impact Hardness (RIH), Coefficient of Rock Strength (CRS), and the Swedish Brittleness Test (S_{20}), which is incorporated in the prediction method developed by the Norwegian Institute of Technology (NTH). Many "drillability" and "abrasivity" index tests have also been developed; each requires specialized equipment. The CERCHAR (the Laboratoire du Centre d'Études at Recherches des Charbonnages de France) test has been used in assessing abrasivity, and mineralogical abrasiveness measures, including quartz content and Moh's hardness scale, are used.

c. Rock strength testing.

(1) Empirically derived prediction equations have also incorporated results from "conventional" rock strength testing. The rock property most widely used in performance prediction has been the uniaxial compressive strength (UCS) primarily because of the availability of UCS test results. However, UCS may not be the ideal parameter for TBM performance prediction unless in situ variability of UCS (or of index test results) is evaluated.

(2) Rock tensile strength, most often measured in a Brazil test, may also be used for machine performance prediction. Test results can be used for weak rock to evaluate whether brittle behavior will occur on disc indentation and to evaluate rock strength anisotropy.

(3) Rock fracture toughness and other fracture material properties (such as the critical energy release rate or critical crack driving force) have great potential application for machine performance prediction. However, few tests have been performed at tunneling projects so the correlations performance demonstrated to date must be considered preliminary.

(4) Other descriptive properties are also evaluated during site investigations, and many empirical correlations have included these in linear regression equations. Such properties include density, porosity, water content, and seismic velocities. For weak rock, Atterberg limits and clay mineralogy should be evaluated early in the site investigation, with more specialized testing for swell, squeeze, and consolidation properties perhaps warranted on the basis of the results of index tests.

(5) At this time, a recommended suite of rock property tests for tunnel project investigations should include both tensile and compressive strength, an evaluation of porosity or other measure of dilative versus compactive response, and an evaluation of rock abrasivity. Care should be taken with the core to minimize stress-relief effects and moisture loss. Sampling biases for or against very weak or very strong rock must be avoided, because it is these extremes that often define success or failure for a TBM application. For use in specific predictive approaches, particular tests can be performed, such as the various hardness tests or the suite of tests incorporated into the NTH methodology. In all cases, specified equipment for index property testing is mandatory, and suggested procedures must be followed. Guidance concerning required testing can be sought from TBM designers and consultants.

e. Empirical equations.

(1) Three commonly applied performance correlations using empirical equations developed from data on rock testing are presented below, with $PRev$ evaluated in units of millimeters/revolution, F_n in kN, and the compressive (UCS) and Brazilian tensile (σ_{tB}) strengths expressed in units of MPa or kPa, as noted.

(2) Farmer and Glossop (1980), who include mostly sedimentary rocks in their database, derived the following equation:

$$PRev = 624\ F_n\ /\ \sigma_{tB} \tag{C-4}$$

(3) Graham (1976) derived a similar equation that uses UCS for a predominantly hard rock (UCS 140 to 200 MPa) database:

$$PRev = 3940\ F_n\ /\ UCS \tag{C-5}$$

(4) Hughes (1986) derived a relationship from mining in coal:

$$PRev = 1.667\ (F_n/UCS)^{1.2} \cdot \left(\frac{2}{D}\right)^{0.6} \qquad \text{(C-6)}$$

where D is the disc diameter in millimeter, and it is assumed that only one disc tracks in each kerf groove, the normal practice for TBM design.

e. Performance data.

(1) Rock properties and machine performance data for three tunnel projects in sedimentary rock are used to demonstrate the predictive ability of these correlations in Table C-1. Rock test results, TBM performance, and predicted penetration rates are shown in the table. Average disc forces vary directly with UCS, and the maximum load is well below the maximum load suggested for the cutters used. In each case, TBM penetration and thrust were limited by available torque or by the muck handling system capacity.

(2) The predicted penetrations are nearly always less than achieved by TBMs in operation. The Farmer and Glossop equation yields consistently higher predicted penetrations, and the Graham predictions are consistently lowest. The influence of rock test material condition is indicated by the information for the Grimsby Sandstone. Much of the original testing on this project was performed on air-dry rock. When the rock was resaturated and tested, strength reduction was evident. This uncertainty as to intact strength can clearly exert a strong influence on the penetration rate predicted.

(3) The number of equations available leads to an apparent uncertainty in $PRev$ predictions. Such correlations in the public domain have generally been derived from limited databases, and caution against indiscriminant application is recommended. In general application, no single approach can be recommended; rather, use of several equations can be useful to assist in design and selection of equipment and for sensitivity studies of the relative importance of various factors. Thrust forces should, in any event, be increased by 15 to 20 percent for TBM design capacity determination.

f. Cutting coefficients.

(1) Similar equations to predict F_r are not common, largely because while thrust is often monitored during mining, drive motor amperage draw and cutterhead rpm if variable is not often recorded. The approach taken is instead to predict the cutting coefficient, C_c, the ratio of rolling to normal average force. This ratio varies within a general range of 0.1 to 0.25 and is higher for weaker rock, higher $PRev$, and for higher F_n, since F_r tends to increase faster than F_n with increasing $PRev$. C_c can be predicted as a function of $PRev$ and disc diameter only, with the influence of rock strength implicit in the achieved $PRev$.

(2) Roxborough and Phillips (1975) assumed $PRev$ equal to the depth of indentation or cut and derived the following equation for C_c:

Table C-1
Comparison of TBM Case Study and Predicted Penetration Rates

Project Information[1]		Rock Strength (MPa)[2]		TBM Performance		Prediction Method 1-Farmer/ Glossop, 2-Graham, 3-Hughes		
Location	Rock Unit	UCS	Brazil Tensile	F_n, kN	P/rev, mm	1 P/rev	2 P/rev	3 P/rev
Buffalo (NY)	Falkirk Dolostone	188	13.3	134	7.6	6.3	2.8	2.9
	Oatka Dolostone	139	13.0	108	10.4	5.2	3.1	3.3
Rochester (NY)	Williamson/Sodus Shale	80	(8.0)	99	10.0	-	4.9	5.7
	Reynales Limestone	128	15.0	141	6.8	5.9	4.3	5.0
	Maplewood Shale	68	(6.8)	98	10.4	-	5.7	6.8
	Grimsby Sandstone:							
	Wet	130	10.1	112	7.9	6.9	3.4	3.7
	Dry	208	6.1			11.5	4.1	4.6
Chicago (IL)	Romeo Dolostone	237	17.0	145	8.0	5.3	2.4	2.4
	Markgraf Dolostone	168	12.1	137	9.3	7.1	3.2	3.5
Austin (TX)	Austin Chalk	10	1.3	33	9.6	15.7	99.1	18.5

[1] Sources: NY and IL projects (Nelson 1983), TX project (Hemphill 1990).
[2] (8.0) and (6.8) for Brazil tensile strength are estimated as UCS/10.

$$C_c = F_r/F_n = \sqrt{PRev/(D - PRev)} \qquad \text{(C-7)}$$

(3) An equation adopted in Colorado School of Mine's predictive method (Ozdemir and Wang 1979) is:

$$C_c = \tan(\phi/2);\ \phi = \cos^{-1}[(R - PRev)/R], \qquad \text{(C-8)}$$

which is actually the Roxborough and Phillips equation in different form. Hughes (1986) suggests:

$$C_c = 0.65\sqrt{PRev/(D/2)} \qquad \text{(C-9)}$$

In these equations, D is the disc diameter and R is the disc radius. Table C-2 records the results of an equation comparison for 432-mm-diam cutters. The similarity of the results is clear and either can be used to predict C_c and hence F_r and required power for a selected cutterhead rpm.

Table C-2

PRev, mm	Roxborough and Phillips/CSM	Hughes
4	0.10	0.09
8	0.14	0.13
12	0.17	0.15

C-3. TBM Performance Prediction via Linear Cutter Testing

a. A direct way to determine force requirements for TBM design is to perform laboratory linear cutting tests with the rotary TBM cutting process modeled as linear paths of indexed cutter indentations. Linear cutter testing has been used by contractors who plan to make their own decisions about equipment purchase or reconditioning. Such testing is expensive and not likely to be pursued for all tunnel projects. Linear cutter test results of cutter force and penetration relationships may be directly applicable to full-scale TBM penetration rate prediction. However, differences between the tested rock and the rock mass in situ, including differences in relative stiffness between the rock mass and TBM, must be considered.

b. Linear cutter test equipment is available at the Earth Mechanics Institute (EMI) of the Colorado School of Mines (CSM). CSM has developed a complete prediction method for TBM performance using field values of operating thrust, torque, cutter type, and spacing. The predictions are consistent with actual performance except when applied directly to TBM use in blocky or jointed rock masses. A match of disc cutter tip width and diameter between the field and linear cutter testing is important for accurate predictions of both forces and penetration.

C-4. Impact of Rock Mass Characteristics on TBM Performance Prediction

a. Impact of rock mass characteristics.

(1) Rock mass characteristics impact penetration rate in several ways. For example, see below:

(a) If a mixed face of variable rock strength is present at the heading, the penetration rate is more typical of the stronger rock.

(b) For good rock, penetration rate will increase as more discontinuities are present at the face. Penetration rates will be greater when discontinuities are oriented parallel to the rockface.

(c) If rock condition deterioration by geologic structure or weathering is severe, TBM thrust and torque may be reduced to promote face stability.

(2) These factors can be used to guide site investigation efforts. For example, in the common situation of flat-lying sedimentary rock, RQD determined on vertical exploratory core cannot supply information on the frequency of vertical discontinuities that can be exploited in the process of chip formation and are important for penetration rate prediction.

(3) The same factors are generally true of intact rock anisotropy, which can greatly enhance penetration rates, depending on orientation with respect to the tunnel face. Anisotropy effects may be included implicitly in intact rock prediction methods by controlling rock specimen orientation during testing. Tests such as Brazil tension and point load tests have been used for this purpose. On a larger scale, a similar effect can occur, as long as discontinuity frequency does not significantly increase rock support requirements. Increased jointing permits *PRev* increase at decreased F_n, perhaps doubling *PRev* when joint spacings approach cutter spacing. The effect is most important for thrust-limited mining in stronger rock.

b. Ground difficulty index.

(1) Eusebio et al. (1991) introduced a "Ground Difficulty Index" (GDI) classification scheme, developed from data for a tunnel driven in highly variable rock. Rock

mass RQD and RMR classifications were determined, and in situ Schmidt hammer testing was used to measure intact rock strength variability. From a "basic" penetration rate derived empirically from UCS and including the effect of F_n on penetration, an empirical multiplier (f1) on *PRev* can be identified depending on RMR classification, as shown in Table C-3:

Table C-3

RMR Class	f1
I	1.0
II	1.1
III	1.1-1.2
IV	1.3-1.4
V	0.7

(2) A similar approach has been taken by Casinelli et al. (1982), who suggest a correlation between specific energy (SE, in kilowatt hours/cubic meter) and RSR, based on tunnel excavation in granite gneiss as:

$$SE = 0.665\ RSR - 23 \qquad \text{(C-10)}$$

for RSR > 50, with RSR the Rock Structure Rating.

(3) The EMI at the CSM has developed an equation to evaluate rock mass impacts based on RQD. Using a database for weaker rocks (UCS < 110 MPa), CSM recommends a multiplying factor, F1, to modify a basic *PRev* determined for "perfect" RQD = 100 rock as:

$$F1 = 1.0 + (100 - RQD) / 150 \qquad \text{(C-11)}$$

and for stronger rocks (UCS $\geq$ 110 MPa) as:

$$F1 = 1.0 + (100 - RQD) / 75 \qquad \text{(C-12)}$$

The increased importance of jointing in stronger rock is evident in these equations.

c. *Impact of in situ stresses.*

(1) In situ stresses that are high relative to rock strength can promote stress slabbing at the face. At typical mining rates, this response may result in an increased *PRev* if the rock is not greatly overstressed or susceptible to bursting. However, face deterioration and overbreak may develop, which must be controlled with shielding or cutterhead modifications such as false-facing in severe cases. In fact, the TBM operator usually decreases F_n and cutterhead rotation rate to improve face stability.

(2) To summarize, if rock support requirements are not changed significantly, a penetration rate (PR) increase can be expected with increased jointing present in a rock mass. Such an effect is most important to consider in very strong rock for which modest increases in PR can significantly improve the economics of a project. In practice, any PR improvement is either implicitly included within empirical correlations or ignored, in anticipation that the impact of any rock instability will dominate the performance response.

(3) As indicated in the summary presented in Table C-4, the primary impact of rock mass properties on TBM performance is on utilization, an impact that depends greatly on chosen equipment and support methods. Site investigations should be geared to address certain basic questions for equipment selection. In weak rock, mucking and rock support are major downtime sources; in very strong rock, equipment wear at high loads and cutter wear are often the major downtime sources. In either case, correct appreciation of the problem or limitation before the equipment is ordered goes a long way toward minimizing the geotechnical impacts. The actions and decisions associated with the answer to each geomechanics question are often the responsibility of the contractor, but clear assessment of each geomechanics question is the responsibility of the investigating engineers.

C-5. Impact of Cutting Tools on TBM Performance

The primary impact of disc wear is on costs that can be so severe that cutter costs are often considered as a separate item in bid preparation. The UT database indicates that about 1.5 hr are required for a solitary cutter change, and if several cutters are changed at one time, perhaps 30 to 40 min are required per cutter. Higher downtime is closely correlated with large ground water inflows, which make cutter change activities time-consuming. Disc replacement rates vary across the cutterhead, with low rolling distance life associated with center cutter positions where tight turning and scuffing reduce bearing life and vibrations can cause particularly high rates of abrasive wear. For relatively nonabrasive rock, rolling distance life for cutters in gage and face positions are comparable. However, gage replacement rates are higher in terms of TBM operating time because the travel path is longer and the cutters "wash" through muck accumulations. Gage cutter rolling

Table C-4
Impacts of Geotechnical Conditions on TBM Operations

Major Geotechnical Conditions	Consequences/Requirements
Loosening loads, blocky/slabby rock, overbreak, cave-ins	At the face: cutterhead jams, disc impact loading, cutter disc and mount damage possible, additional loss on available torque for cutting, entry to the face may be required with impact on equipment selection, recessed cutters may be recommended for face ground control. In the tunnel: short stand-up time, delays for immediate and additional support (perhaps grouting, hand-mining), special equipment (perhaps machine modifications), gripper anchorage and steering difficulty, shut-down in extreme cases of face and crown instability. Extent of zones (perhaps with verification by advance sensing/probe hole drilling) may dictate shield required, and potential impact on lining type selection (as expanded segmental linings may not be reasonable), grouting, and backpacking time and costs may be high.
Groundwater inflow	Low flow/low pressure - operating nuisance, slow-down, adequate pumping capability high flow and/or high pressure - construction safety concerns, progress slow or shut-down, special procedures for support and water/wet muck handling, may require advance sensing/probe hole drilling. Corrosive or high-salt water - treatment may be required before disposal, equipment damage, concrete reactivity, problems during facility operation. Equipment modifications (as water-proofing) may be required if inflow is unanticipated - significant delays.
Squeezing ground	Shield stalling, must determine how extensive and how fast squeeze can develop, delays for immediate support, equipment modifications may be needed, if invert heave and train mucking - track repair and derail downtime.
Ground gas/hazardous fluids/wastes	Construction safety concerns, safe equipment more expensive, need increased ventilation capacity, delays for advance sensing/probing and perhaps project shut-down, special equipment modifications with great delays if unanticipated, muck management and disposal problems.
Overstress, spalls, bursts	Delays for immediate support, perhaps progress shut-down, construction safety concerns, special procedures may be required.
Hard, abrasive rock	Reduced *PRev* and increased F_n - TBM needs adequate installed capacities to achieve reasonable advance rates, delays for high cutter wear and cutterhead damage (especially if jointed/fractured), cutterhead fatigue, and potential bearing problems
Mixed-strength rock	Impact disc loading may increase failure rates, concern for side wall gripping problems with open shields, possible steering problems.
Variable weathering, soil-like zones, faults	Slowed progress, if sidewall grippers not usable may need shield, immediate and additional support, potential for groundwater inflow, muck transport (handling and derails) problems, steering difficulty, weathering particularly important in argillaceous rock.
Weak rock at invert	Reduced utilization from poor trafficability, grade, and alignment - steering problems.

distance life is notably reduced in highly abrasive rock mining. Database information indicates that TBM penetration rate is generally unaffected by disc cutter abrasion until the wear causes about a 40-mm decrease in disc diameter. For additional amounts of wear, penetration rate may only be maintained with increased F_n. If thrust is not increased, the penetration rate achieved may be reduced by 15 to 25 percent. Normal cutterhead maintenance checks will guard against this happening. It is particularly important for the contractor to develop a management plan to promote cutter life, since high cutter loads associated with worn cutters can result in higher disc and bearing temperatures and in more bearing and seal failures. Regular inspection and planned replacements are required to maximize disc life, reduce cutter change downtime, and minimize cost and schedule impacts. Cutter change downtime can also be expressed on the basis of shift time. For nonabrasive rock, the cutter downtime may be on the order of 3 percent. For highly abrasive rock, however, cutter changes may require more than 20 percent of all shift time. Cutter change downtime can also be recorded as hours required per meter of excavation. For nonabrasive rock, average cutter change downtime was 0.02 to 0.05 hr/m. For more abrasive rock, downtime may increase to more than 0.2 hr/m. Tight alignment curves can decrease cutter disc life significantly. The EMI at the CSM has developed an equation to evaluate alignment curve radius impacts on cutter life. CSM recommends a multiplying factor, *F*2, to

modify an expected "normal" cutter life for alignment curves of radius R, in meters determined for "perfect" RQD = 100 rock as:

$$F2 = 1.0 - 23/R \tag{C-13}$$

The recent trend toward larger disc diameter means that cutters are heavier, and equipment must be installed to facilitate cutter transport and installation. Wedge-lock housing has been developed that makes cutter changes much easier and that has proven to be very durable. Other improvements include rear-access cutters that do not require access to the front of the cutterhead for replacement. In cases of face instability, these cutters greatly improve safety but are more expensive and take more time to replace.

In abrasive conditions, significant wear of the cutter mount and hub can occur with reduced disc bearing life. In relatively nonabrasive rock, 6 to 10 discs can be refit on each hub before repair is necessary. However, in abrasive sandstone, a rate of only 1 to 3 discs per hub may be typical. In very abrasive rock, tungsten carbide cutters may be used at increased expense. Most of the databases on cutter replacement rates and costs are proprietary. The largest public-domain database for abrasive wear rate prediction can be accessed through the NTH (1988) method, but specific rock tests must be performed that require special equipment. If abrasive conditions are anticipated, it is important to submit samples for testing by machine manufacturers, contractors, and specialized consultants.

C-6. The EMI TBM Utilization Prediction Method

a. Several databases can be accessed to assist in evaluations of TBM utilization. In the future, a complete simulation computer program including all components of TBM construction operations will be available through the Texas database analysis.

b. The EMI CSM (Sharp and Ozdemir 1991) also has developed an approach to evaluate TBM utilization via analysis of a proprietary database. To account for delays associated with thrust cylinder piston restroke, a parameter $F3$ is recommended as:

$$F3\ (hr/m) = 0.030\ (hr/m) + (409\ m\text{-}hr)/R^2 \tag{C-14}$$

where R is the radius of alignment curvature in meters. For straight tunnel sections, this equation predicts about 2.7 min per 0.45-m stroke cycle. For tight curves of perhaps 150-m radius, this stroke reset time increases to 4.4 min. To account for unscheduled maintenance and repairs, a factor $F4$ (in units of delay hours) is evaluated as:

$$F4\ during\ start\text{-}up = 1.0\ hr\ per\ TBM\ mining\ hr$$

and

$$F4\ following\ start\text{-}up = 0.324\ hr\ per\ TBM\ mining\ hr.$$

c. The start-up period is identified as a learning curve with shift utilization deceasing to a fairly constant value corresponding to production mining. Scheduled maintenance, including cutterhead checks and TBM lubrication, should be evaluated at 0.067 delay hours per TBM mining hour.

d. Surveying delays are discretely accounted for in the CSM approach. Normal delays for straight tunnel sections are minimal at 0.0033 hr per meter of bored tunnel. For alignment curves, survey delays are evaluated as:

$$Survey\ delay\ (hr/m) = 0.0033 + 192\ m\text{-}hr\ /\ R^2 \tag{C-15}$$

where R is the radius of curvature in meters. For a 150-m-radius curve over a 200-m-long tunnel length, survey delays of about 2.5 hr should be expected by this equation.

e. For minimal nuisance water inflows, delays can be expected at a rate of about 0.0056 hr per meter of bored tunnel. For conditions of inflow up to about 3 to 4 m^3/min/m of tunnel, delays on the order of 0.085 hr/m of bored tunnel should be expected. Excess water inflow and grouting precipitates additional delays that are higher for increasing inflow volumes and for low gradient to downhill tunnel driving. For example, for downhill grades, delays will multiply to 2 hr/m of tunnel at inflow rates in excess of 13 to 15 m^3/min/m of tunnel.

f. Delays associated with the tunnel mucking system can be estimated considering tunnel gradient, direction of drive, and expected mucking system. Table C-5 shows some general guidelines.

Table C-5

Tunnel Description	Mucking Method	Delay hr/m
Start-up Driving	Trucks	0.115
Production Driving		
-15° to -1° down	Conveyor	0.071
-1° to +3°	Train	0.056
+3° to +15° uphill	Conveyor	0.071

Delays associated with extending utility lines will also depend on tunnel grade:

$$\text{Utility Delays (hr/m of tunnel)} = 0.030 + 0.0013\,G \tag{C-16}$$

with G the tunnel grade defined as the angle (in degrees) of TBM driving above (>0) or below (<0) the horizontal. Delays associated with installing temporary support accumulate as a function of rock mass quality. In the CSM approach, Rock Support Category (RSC), similar to the classes resulting from RMR classification, is used. See Table C-6. Labor delays are evaluated to cover time spent on shift changes, safety meetings, lunches, etc. CSM recommends using 2.5 percent of the overall shift time as labor-delay downtime.

Table C-6

RSC Category	Delay (hr/m of bored tunnel)
I	0
II	0
III	0
IV	0.028
V	0.043

g. The CSM approach includes all aspects of TBM operations, and its validity for general application resides in the proprietary database used to derive these equations. However, the cutter life and *PRev* prediction methods are not in the public domain. Until more data analysis is completed in the public domain, however, the CSM methodology is recommended as a way to evaluate decisions required for project alignment and equipment selection.

C-7. The NTH TBM Performance Prediction Methodology

a. The Norwegian Institute of Technology (NTH) has developed the most thorough published predictive approach for TBM performance (NTH 1988). The NTH method is certainly the most systematic method available in public domain and includes all desirable aspects of TBM design and operation, including thrust, torque, rotation rate, cutterhead profile, disc spacing and diameter, and disc bluntness.

b. Intact rock tests required in the methodology include three specialized tests for abrasivity value (AV), brittleness (S_{20} from the Swedish Brittleness test), and drillability (the Sievers J Value). Derived rock parameters include the Drilling Rate Index (DRI) and Cutter Life Index (CLI). The F_n versus *PRev* relationship is nonlinear, and the concept of "critical thrust" is incorporated as a normalizing parameter. Various factors are offered to modify the calculated *PRev*, thrust, and torque for differences in cutter diameter and kerf spacing.

c. The NTH method is derived for a database consisting primarily of experience in Scandinavian rocks and may be considered more suitable for application to tunneling in igneous and metamorphic rock. Certain "rules" for TBM design are also incorporated into the figures presented:

- Cutterhead rpm is established by maximum gage cutter rolling velocity (Table C-7):

Table C-7

Disc Diameter		Max. Gage Velocity
mm	in.	m/min
356	14	100
394	15.5	120
432	17	160

- Disc groove average spacing (TBM radius/number of discs), assuming only one disc cutting each groove, is set at about 65 mm.

- Maximum cutter loading is dependent on disc diameter (Table C-8):

Table C-8		
Disc Diameter		Max. Disc Cutter Load
mm	in.	kN
356	14	140-160
394	15.5	180-200
432	17	220-240
483	19	280-300

- Installed cutterhead power is expected according to the relations shown in Table C-9:

Table C-9		
Cutter Diameter		Installed Power
mm	in.	kW
356	14	700 + 140 (D - 5 m)
394	15.5	850 + 170 (D - 5 m)
432	17	1,050 + 200 (D - 5 m)
483	19	1,800 + 360 (D - 5 m)

d. The method for *PRev* prediction relies on DRI values that can be tested through NTH, although correlations between DRI and UCS (determined on 32-mm-diam cores) are presented for some rock types in Table C-10. Note that low DRI values correspond to difficult drilling, so that low DRI generally corresponds to high UCS.

Table C-10		
Rock	DRI Range	Range in UCS, MPa
Quartzite	20-55	>400-100
Basalt	30-75	
Gneiss	30-50	300-100
Mica Gneiss/ Coarse Granite	30-70	240-70
Schist/Phyllite	35-75	150-50
Med/Fine Granite	30-65	280-120
Limestone	50-80	110-70
Shale	55-85	30-10
Sandstone	45-65	180-100
Siltstone	60-80	100-20

e. The NTH method relies on CLI, the cutter life index for disc replacement rate estimation. The NTH database includes the information on CLI shown in Table C-11:

Table C-11	
Rock	CLI Range
Quartzite	0-8
Basalt	25-75
Gneiss	2-25
Schist/Phyllite	10-40
Med/Fine Granite	30-65
Limestone	70 to >100
Shale	40 to >100

For specific rock types encountered on TBM projects, samples should be submitted to NTH for CLI evaluation.

f. The NTH approach to TBM performance estimation, summarized herein, represents a discussion of the general methodology. The many figures and tables included in the source manual are reduced to close approximations for presentation in this document. If precise values of the identified factors are desired, the user should consult the NTH project report.

g. In the NTH method, the *PRev* prediction is achieved as:

$$PRev = [F_n/M_1]^b \tag{C-17}$$

with M_1 found as a "critical thrust," evaluated for $PRev$ = 1 mm, and b is the "penetration coefficient."

The M_1 is found from a sequence of figures in the NTH report and is a function of DRI and factors associated with disc diameter (k_d), disc groove spacing (k_a), and rock mass fracturing (k_s). The k_s factor effectively modifies the thrust versus penetration relationship for a given intact rock, such that the more fractured a rock mass is, the higher the *PRev* achieved for a given F_n. This factor is also used in torque calculations since, in fractured rock, torque demand increases with increased penetration. The M_1 increases with increasing cutter diameter and spacing and decreases with higher DRI and increased fracturing (high k_s).

The k_d factor is found as shown in Table C-12:

Table C-12		
Disc Diameter		
mm	in.	k_d
356	14	0.84
394	15.5	1.00
432	17	1.18
483	19	1.42

The k_a factor can be approximately found as:

$$k_a = 0.35 + s / 100 \tag{C-18}$$

where s is the average disc spacing, in millimeters. The k_s factor is a function of a classification made on the basis of spacing and strength of discontinuities (joints or fissures) present in a rock mass. Joints are defined as discontinuities that are open, or weak if filled, and continuous over the size of the excavation. Fissures generally include bedding and foliation—discontinuities with somewhat higher strength than joints. If a rock mass contains no discontinuities, or those present are filled or healed so as to be of very high strength, the material is considered massive rock (Class 0). Table C-13 indicates the general range of k_s expected for rock masses dominated by various classes of jointing or fissuring. The low end of each k_s range corresponds to discontinuities generally trending normal to the excavated face or with strike parallel to tunnel axis. The high end range of k_s corresponds to discontinuities favorably oriented for chip formation, i.e., parallel to the excavated face or with relative strike perpendicular to the tunnel axis. Users of the NTH method should consult the referenced manual for a complete treatment of k_s selection. For joints at close spacing, it is likely that face instability will dominate TBM operations, and no k_s is assigned.

h. In the NTH database, Class 0 - I rocks were generally gneiss, quartzite, and basalt. Classes III and IV are predominantly populated by schists, phyllites, and shales. The penetration coefficient, b, is found as a function of M_1, disc spacing, and disc diameter. The coefficient varies from about 1.0 to greater than 4.0; b is highest for large M_1 values and disc diameter, and more closely spaced cutter grooves or, in general, for stronger rock. Correct selection of b is very important to the NTH approach as it is the exponent used to establish the basic force/penetration relationship. Reference should be made to NTH for appropriate rock testing and selection of both M_1 and b for site-specific applications. With all parameters identified, it is possible to evaluate *PRev* and PR, the penetration rate in terms of meter/mining hour, and to design a TBM for required thrust and *PRev*.

i. To evaluate torque requirements, the NTH method uses the following equation:

$$F_r = F_n \sqrt{PRev} \tag{C-19}$$

where C is the cutter constant, a function of disc diameter, k_s, and cutter sharpness. In application, the NTH method sometimes has indicated lower penetration rates than were achieved. This difference is due to the method being based upon laboratory test results and not in situ strengths. The NTH methodology includes an approach to estimate cutter replacement rates. The prediction is based on the Cutter Life Index (CLI), a compound parameter depending on the Abrasion Value (determined for steel rings) and the Siever's J-value (a drillability test).

j. Average disc life, L_h, in units of TBM mining hours per cutter, is found as:

$$L_h = DL\ k_\phi\ k_{rpm}\ k_N\ k_{min} / N \tag{C-20}$$

Table C-13				
Joints		Fissures		k_s
Class	Spacing	Class	Spacing	
0	>1.6 m	0	>1.6 m	0.36
0-I	≈ 1.6	I	0.8-1.6	0.5-1.1
I	0.8-1.6	II	0.4-0.8	0.9-1.5
I-II	0.4-0.8	II-III	0.2-0.4	1.1-1.8
II	0.2-0.4	III	0.1-0.2	1.3-2.3
II-III	0.1-0.2	III-IV	0.1-0.05	1.9-3.0
>III	not valid	IV	<0.05	3.0-4.4

Disc Diameter		K_s Range	C	
mm	in.		blunt	sharp
356	14	from <0.75	0.038	0.044
		up to ≈4.0	0.070	0.082
394	15.5	from <1.0	0.034	0.041
		up to ≈4.0	0.050	0.060
432	17	all	0.025	0.033
483	19	all	0.018	0.027

where N is the number of discs, and DL is the "Disc Life," found as shown in Table C-14:

Table C-14

Disc Diameter		
mm	in.	DL, TBM hrs
356	14	8.6 CLI
394	15.5	12.4 CLI
432	17	17.4 CLI
483	19	26.3 CLI

k. The various correction factors are defined as follows. The correction factor k_ϕ is a correction for TBM diameter and cutterhead type, required since the proportion of gage cutters decreases as TBM diameter increases, and because cutters on flat-faced cutterheads have longer life than do cutters on domed cutterheads. Values for k_ϕ are shown in Table C-15.

Table C-15

	k_ϕ	
TBM Diameter, m	Domed	Flat
3	0.92	1.04
5	1.19	1.34
7	1.40	1.58
10	1.67	1.87

(2) The correction factor k_{rpm} is for cutterhead rotation rate, required since the faster the rpm, the higher the rolling velocities and the shorter the disc life. This correction factor is found as

$$k_{rpm} = 38/(D\ rpm) \qquad \text{(C-21)}$$

where rpm is the cutterhead rotation rate in revolutions per minute and D is the diameter of the TBM in meters.

(3) The correction factor k_N is developed for TBMs where disc spacing is not at the 65 mm assumed. With more discs at smaller spacing, a longer life is expected. If s is the average disc spacing in millimeters (TBM radius divided by the number of cutters), k_N is found as

$$k_N = 65/s \qquad \text{(C-22)}$$

The correction factor k_{min} is designed to correct the estimated cutter life for the presence of abrasive minerals such as quartz, mica, and amphibole. This correction factor is calculated as:

$$k_{\min} = k_{quartz}\ k_{mica}\ k_{amph} \qquad \text{(C-23)}$$

with the correction factors for individual minerals found to sufficient accuracy by interpolation from values in Table C-16 with the mineral content defined on a volume percent basis:

Table C-16

Mineral Content, Volume %	k_{quartz}	k_{mica}	k_{amph}
0	1.0	1.0	1.0
10	0.74	0.78	0.90
20	0.67	0.72	0.58
30	0.65	0.67	0.46
40	0.65	0.65	0.38
50	0.65	0.62	0.34
≥60	0.65	0.60	0.31

l. Using results from *PRev* calculation, it is also possible to express cutter life in terms of cutter rolling distance or cubic meters of rock excavated per cutter change. By the NTH database, typical 394-mm-diam rolling distance life varies from 200 to 1,000 km for highly abrasive rock, and up to 5,000 to 10,000 km for nonabrasive rock. Cutter life is reduced by 30 percent for 356-mm-diam cutters and increased by 50 to 65 percent for 432-mm-diam cutters. Cutters on flat cutterheads have 10-percent longer life than on domed cutterheads, and constant section cutters last 10 to 15 percent longer than do wedge section cutters with similar amounts of steel in the disc rings. Mining around tight curves reduces cutter life by about 75 percent.

m. The NTH methodology also permits utilization and advance rate prediction in a manner similar to that used in the CSM approach as outlined below:

- The mining time, T_b, can be evaluated from the *PRev* established previously.

- Regrip time, T_t, estimated as about 5.5 min per reset cycle.

- The cutter change downtime, T_k, is estimated using the output from cutter life calculations. For cutter diameters $\leq$ 432 mm (17 in.), NTH suggests using 45 min per cutter change. For larger cutters, a suggested 50 min per change should be used.

- The TBM maintenance downtime, T_{TBM}, is estimated as 150 shift hours per kilometer of mined tunnel.

- The time required for maintenance and repair of backup systems, T_{bak}, is estimated from the table below.

- Miscellaneous downtime, T_a, includes other activities as waiting for return of empty muck cars, surveying, electrical installations. The T_a is related to type of back-up equipment and can also be estimated from information in Table C-17.

Table C-17

	Shift hr/km mined tunnel	
Back-up System	T_{bak}	T_a
Single track	40	185
Double track	90	95
Trackless	55	95

The sum of these time increments equals the shift time, from which utilization and advance rate can be calculated. The NTH method also includes approaches to evaluate project cost, support requirements, and additional information on all components of downtime, site investigations, and interpretation of geologic conditions.

Appendix D
Conversion Factors

Physical Quantity		
Length	1 in. = 25.4 mm 1 ft = 0.3048 m 1 mile = 1,609.344 m	39.3701 in. 1 m = 3.28084 ft $0.621371 \cdot 10^{-3}$ mile
Area	1 in.^{2} = 645.16 mm^2 1 ft^2 = 0.092903 m^2	1,550 in.^{2} 1 m^2 = 10.7639 ft^2
Volume	1 ft^3= 28.3168 1 yd^3 = 0.764555 m^3 1 U.S. gal = 3.78541	$35.3147 \cdot 10^{-3}$ ft^3 1 = $1.30795 \cdot 10^{-3}$ yd^3 0.264172 U.S. gal
Mass	1 lb = 0.453592 kg 1 ton U.S. = 907.185 kg	2.20462 lb 1 kg = $1.10231 \cdot 10^{-3}$ ton U.S.
Density	1 lb/ft^3 = 16.0185 kg/m^3 1 lb/in.^{3} = 27,679.9 kg/m^3	0.0624278 lb/ft^3 1 kg/m^3 = $36.127 \cdot 10^{-6}$ lb/in.^{3}
Force	1 kgf = 9.80665 N 1 lbf = 4.44822 N	0.101972 kgf 1 N=0.224809 lbf
Moment (Energy)	1 kgf · m = 9.80665 Nm 1 lbf · ft = 1.35582 Nm	0.101972 kgf · m 1 Nm=0.737562 lbf · ft
Pressure or stress	1 bar = 100 kPa 1 psi = 6.89476 kPa 1 psf = 0.04788 kPa	0.01 bar 1 kPa = 0.145038 psi = 20.89 psf
Energy	1 kWh = 3.6 MJ 1 hkh = 2.6478 MJ 1 hkh = 0.735 kWh	$0.277778 \cdot 10^{-3}$ kWh 1 kJ = $0.377673 \cdot 10^{-3}$ hkh 1 kWh = 1.356 hkh
Flow	1 gpm = 0.063090 l/sec 1 cfm = 0.028317 m^3/min	1 l/sec = 15.8503 gpm 1 m^3/min = 35.315 cfm

SI-unit prefixes

Prefix	tera	giga	mega	kilo	milli	micro	nano	pico
Multiplier	10^{12}	10^9	10^6	10^3	10^{-3}	10^{-6}	10^{-9}	10^{-12}

Note: N = Newton = kg · m/sec^2, J = Joule = Nm, T = Tonne = 10^3 kg, W = Watt = J/sec, g = gram = 10^{-3} kg, Hz = Hertz = cycles/sec